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(INSTITUTED 1852.)

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No. 833.

FLUSHING IN PIPE SEWERS.

By H. N. OGDEN, Jun. Am. Soc. C. E.

PRESENTED MAY 4TH, 1898.

WITH DISCUSSION.

The use of flush-tanks in connection with small pipe sewers, which has been made an integral part of the "Separate System" and generally adopted in systems caring only for house sewage, is attended with much uncertainty. In such systems it is generally specified that a flush-tank be placed at the head of every lateral, each tank being so regulated as to discharge at least once in 24 hours. The relation between the size of the sewer pipe and the amount of water used in a flush is not given, nor is the influence of grade discussed. The general law is laid down that all laterals, regardless of size, grade, or contributing population, must be supplied with flush-tanks in order to secure a self-cleansing flow in the laterals and to maintain the integrity of the system.

The financial burden of such a requirement is evident. As an example, it may be cited that in the plans for the sewerage system of Ithaca, N. Y., in which this requirement of flush-tanks was thoroughly complied with, even for the 12% grades, no less than 131 flush-tanks were required in 25.3 miles of sewers, or one for every 1 020 ft. The relative importance of the flush-tanks may also be seen by comparing

the actual cost of the sewers with the estimated cost of the tanks. The cost of the sewers, viz., the sum of the amounts of the several contracts was $81 000, and, estimated at $50.00 each, the flush-tanks would cost $6 550, or more than 8% of the cost of the system. It would seem, then, that the cost of flush-tanks is by no means insignificant, but that their use increases the cost of the separate system by nearly one-tenth, besides introducing a permanent charge, both for water used and for intelligent care in maintenance. That these annual charges are no bagatelle will be apparent by again referring to the case of Ithaca. Assuming that the tanks required are of only 150 galls. capacity, a minimum amount, discharging but once a day, the water required is 19 650 galls. a day. Twenty cents per 1 000 galls. (the amount charged in Ithaca*) is a fair average amount, and at that price the daily charge for water is $3.93 or $1 434.45 per year. Adding to this $600 per year as the wages of a mechanic, whose constant attention is found by experience to be necessary in examining and readjusting the tanks, the total annual charge is $2 034.45. This, capitalized at 6%, gives $33 908, and, added to the $6 550, gives $40 458 as the total expenditure on account of flush-tanks in a sewer system costing for pipe laid $81 000. Surely the item of flush-tanks is an important one, and should be carefully examined, so that if the conditions of the sewer grade, for example, modify the necessity for tanks, or if the amount of water is a function of the time interval between flushes, or of the size of the pipe, it may be known in order that the large proportionate cost of flushing may be reduced to what has been found by careful investigation to be an absolute minimum.

That the requirement given above is felt by present-day engineers to be largely in excess of necessity is sufficiently evident from a study of the paper by F. S. Odell, M. Am. Soc. C. E., entitled "The Separate Sewer System Without Automatic Flush-Tanks,"† and the subsequent discussion, in which the author says that at Mt. Vernon, N. Y., no flush-tanks are used, and that, while hand-flushing by means of fire hose is practiced at intervals of six months, even this infrequent flushing does not appear necessary, as examination of the sewers invariably shows a very wholesome and satisfactory condition. In the discussion very little positive evidence is given, but the experiences recorded go chiefly to show that while automatic flush-tanks do not in them-

* *Manual of American Water Works*, 1897.

† *Transactions*, Vol. xxxiv, page 223.

selves make the separate system practicable, there is, nevertheless, a need, under certain conditions, for flushing, those conditions being as yet not fully determined.

The questions, answers to which are essential for an intelligent disposal of flush-tanks on a sewer system, are four, viz.:

1. What is the relation, if any, between the grade of the sewer and the necessity for automatic flush-tanks?

2. Assuming a need for automatic tanks, how does the grade of the sewer affect the amount of water required, and what is the proper amount to be used?

3. How often should tanks be discharged?

4. What effect does the substitution of a 6-in. for an 8-in. lateral have on the necessity for tanks and on the amount of water to be used?

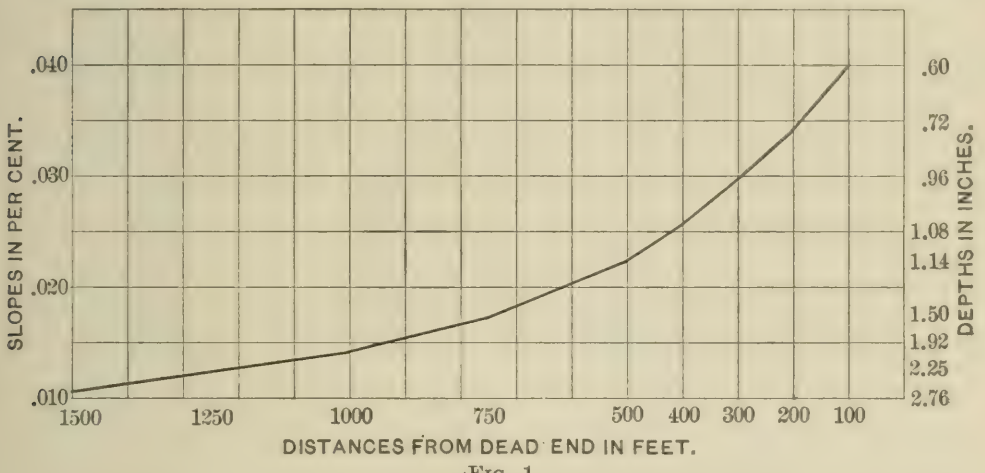


FIG. 1.

Before attempting to answer these questions, it will be well to look at the subject broadly, and consider the hydraulic problem involved. Sewage is water carrying in suspension less than 1 part in 1 000 of solid matter, and sewers are supposed to be so laid that the resulting velocity of flow is sufficient to keep this solid matter in suspension. This suspending and scouring power probably depends on the velocity, and on the depth, of the sewage stream, and if either gets below a certain point, sedimentation will follow and a deposit take place. It is generally stated that a velocity of about $2\frac{1}{2}$ ft. per second is required; but the effect of depth is neglected. At the lower end of a 6-in. lateral the depth and velocity are assumed to be sufficient to prevent this sedimentation, but as the contributing population grows less toward the upper end, the depth and velocity decrease and

the transporting power of the stream falls so low as to allow the solid matter, brought into the sewer by the house drains, to become stranded. This deposit increases by gradual accumulation until the sewer is blocked, until the head from the backed-up sewage is sufficient to carry away the obstruction, or until the discharge of the flush tank (and here is seen its true function) takes up the obstruction and carries it to a point where the depth and velocity of the sewage will hold it in suspension. Table No. 1, and the diagram Fig. 1, are given to show the requirements in grade to maintain a velocity of $2\frac{1}{2}$ ft. per second in a 6-in. lateral, assuming a constant contributing population of 76 persons per 100 ft. of sewer, with a daily flow of 60 galls. per capita, and with the assumption of one-half flowing off in 6 hours.

TABLE No. 1.

Distance from dead end in feet.	Discharge in cu. ft. per sec.	Slope in ft. per foot.	Depth of flow in inches.
1 750	0.245	0.0103	3.00
1 500	0.210	0.0104	2.76
1 250	0.175	0.0123	2.25
1 000	0.140	0.0140	1.92
750	0.105	0.0174	1.50
500	0.070	0.0225	1.14
400	0.056	0.0256	1.08
300	0.042	0.0302	0.96
200	0.028	0.0342	0.72
100	0.014	0.0400	0.60

The diagram (Fig. 1) shows that, taking n equal to 0.013, and computing velocities by Kutter's formula, a grade of 1% is required for a 6-in. pipe half full for a velocity of 2.5 ft. per second, and that if the amount of flow constantly decreases, the depth of flow decreases also, and the grade, in order to maintain the same velocity, must be increased according to the diagram. The diagram is given for two reasons ; first, to show that by the accepted laws governing the transportation of material in flowing water, lateral sewers could be laid, theoretically, on such grades that no flushing would be necessary, since, with grades which continually increase toward the upper end, the corresponding velocities would always be equal to that required to transport matter in suspension ; second, that as the grade of the sewer increases, the distance from the upper end to the point where the stream reaches the velocity required to carry matter in suspension decreases, and so the aid required from flush-tanks is less. No value can be

placed on the grades given, as the diagram is based on the assumption of a house with five persons every 66 ft., and this is not always the case, but it is believed that there is a grade at or beyond which flush-tanks are not required, and that if the distance to which the flushing power extends is a function of the amount of water discharged, then this amount should be less on the steeper grades.

Referring again to Mr. Odell's paper, it is first noted that at Mt. Vernon, with grades of from 0.5% to 6% no flush-tanks are used, and a good hand-flushing twice a year answers every purpose.

In the discussion, Mr. Hering says that on light grades flushes of 200 to 300 galls. generally lose their flushing power after passing a few hundred feet through the pipe, and that sometimes after 500 ft. he had been unable to detect any difference in the flow due to the tank.

Mr. Kiersted writes that in one system designed by him he recommended flush-tanks only on laterals of less than 0.5% grade, and for five years the system has been in operation with but few stoppages.

Mr. Folwell writes that in his experience he has omitted flush-tanks on grades from 6% to 12%, and on the 6% grades no stoppages were discovered, nor were there any odors.

Mr. Le Conte intimates that flush-tanks as built do not answer their purpose, for where grades are light and the flush most needed, they do the poorest work ; and the large quantity of water needed, to be effective, must be obtained by some other means.

Mr. Odell maintains that flushes of 200 galls. or less fail to flush a sewer properly, especially on flat grades where flushing is most needed.

A table by Mr. Allen shows that on grades greater than 0.5% a velocity of more than $2\frac{1}{2}$ ft. per second is maintained over 1 000 ft. from the flush-tank, but on lesser grades the velocity drops to 2 ft. or less within 600 ft.

In order to obtain an insight into general engineering feeling in the matter, and, at the same time, reap the benefit of any experience which was to be had, the author sent out on January 17th, 150 reply postals, reading as follows:

" Ithaca, N. Y., January 17th, 1898.

" Dear Sir:

" To aid me in deciding as to the necessity for flush-tanks for our sewer system, will you kindly answer the following:

"I.—Do you find flush-tanks a necessity, or is periodic hand flushing sufficient to keep sewers clean?

"II.—Does the element of grade affect the question, and within what limits of grade are tanks required?

"III.—Does your experience show any relation between the minimum amount of water required for effective flushing and the grade of the sewer?

"Thanking you in advance for your kind assistance in this matter,

"I am, yours very truly,

"H. N. OGDEN,

"*Engineer, Ithaca Sewer Commission.*"

These postals were sent to those cities of between 10 000 and 60 000 population, in the New England and Middle Atlantic States especially, which were reported in *The Manual of American Water Works for* 1897 as having separate or sanitary sewers. Eighty answers were received, and the courtesy and good-will expressed in all was unmistakable and much appreciated. The same story was told by them in nearly all cases. "I would be pleased to answer your questions fully, but this is the best that I can do for you," or "This is only my idea, while I can readily understand that what you want is the result of actual experience," or "I cannot give you the desired information, but would be thankful to you if you would let me know the result of your inquiry." The results given below in a brief summary chiefly show how uncertain and vague is the knowledge on the subject, and how necessary some experiments and investigations.

In answer to question No. 1, whether flush-tanks were necessary, of the eighty replies seventeen had no opinion on the subject, and twelve had experience only with combined systems, but had, according to their replies, found no trouble in keeping the ends of their 10-in. and 12-in. laterals clean with rain or with hand-flushing. Twenty-six of the eighty used periodic hand-flushing and found it to answer every purpose, keeping the sewers clean and free from obstructions. Twenty-five either used flush-tanks or considered them a necessity for small pipe sewers. It was not possible in these last answers to separate actual experience from personal conjecture on the question, so that this number may include many hearsay opinions.

The evidence is not very clear. The fact that twenty-six used hand-flushing satisfactorily indicates that such flushing is sufficient. That it must be properly and regularly done, however, is made plain

by the fact that, out of the twenty-five believing in flush-tanks, nine had tried periodic hand-flushing, found it uncertain and irregular, and had put in flush-tanks, to secure proper attention. On the other hand, of the twenty-six believing in hand-flushing, two came to that opinion after becoming disgusted with the uncertainty of tanks.

To the second question, only twenty-three of the eighty ventured an opinion. Of these, eight thought that the grade did not affect the question, but that flush-tanks were as necessary on steep as on flat grades. One engineer explained his position by saying that while the velocity on the steep grades might be greater, yet as the depth would be less, the transporting power would be less, and therefore tanks were equally necessary. Of the fifteen who thought that tanks are not needed above a certain grade, six merely ventured it as an opinion, and nine fixed the limit at from 0.5% to 3 per cent. Four give 1% as the limit; one, 3%, and the other four give less than 1 per cent.

Only six replies were given to the last question, whether the amount of water in the flush-tank should be varied with the grade of the sewer. Of these, two thought that no difference should be made; three thought that less water could be used on the steeper grades, but had no definite opinion as to the relative amounts; while one well-known engineer, who has thoroughly studied the workings of the sewer system under his care, writes that he finds one flush daily on a 2% grade as effective as two flushes daily on a 0.5% grade, each flush of 300 galls.

The general conclusion from the replies is that occasional flushing on low grades, probably below 1%, is needed at the upper ends of laterals; that this may be accomplished, either by hand-flushing or by the use of automatic tanks; that if tanks are used, less care and vigilance are required in inspection and oversight, but if they are used, the periodic examination of the system, which should not be omitted, is apt to be irregular, and if a tank fails to work or if an obstruction occurs below the effect of the flush, a serious nuisance may result; that if hand-flushing is used, a constant and regular inspection must be practiced, although actual flushing may be required only once a month or less. The amount of water needed in flush-tanks is not known, nor the relation between amount and grade.

With a view of obtaining more information on this apparently unstudied subject, the author carried on some experiments in the spring

of 1897. He was assisted by Mr. I. W. McConnell, C. E., who had been the writer's valued assistant on the construction of the Ithaca sewer system for two summer vacations. The results of the experiments have been recorded by Mr. McConnell in a thesis for the degree of Civil Engineer in Cornell University.

The sewers on which the experiments were made, and which were chosen so as to afford a variety of grade, with as long lines as possible, were all 8-in. pipe, and each had at the upper end a manhole about 4 ft. in diameter at the bottom. Flush-tanks of usual commercial size discharge at a rate of about 1 cu. ft. per second, and, by repeated experiment, the opening from the manhole into the sewer was reduced to such a size (about 5 ins.) that the rate of discharge varied from 0.89 cu. ft. per second for 4 ft. head in the manhole to 1.1 cu. ft.

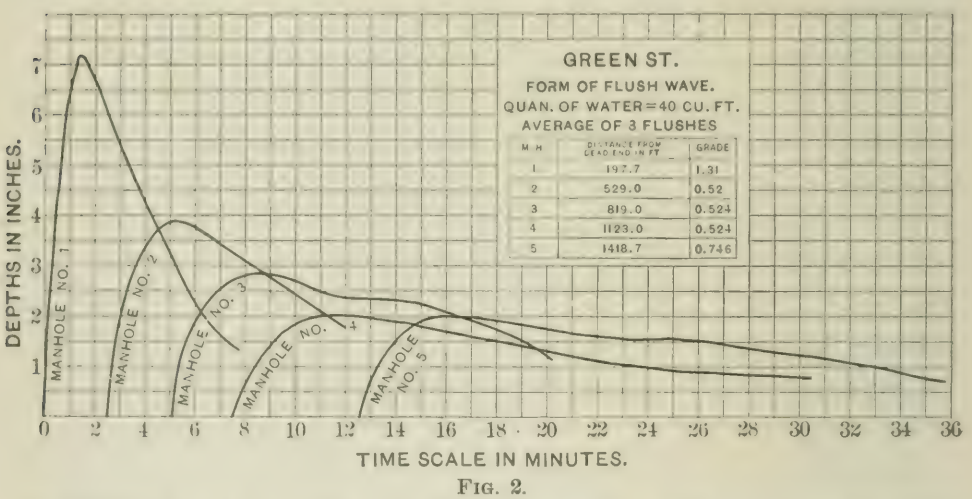

GREEN ST.
FORM OF FLUSH WAVE.
QUAN. OF WATER=40 CU. FT.
AVERAGE OF 3 FLUSHES
M. H.
DISTANCE FROM DEAD END IN FT.
GRADE
1 197.7 1.31
2 529.0 0.52
3 819.0 0.524
4 1123.0 0.524
5 1418.7 0.746
DEPTHS IN INCHES.
MANHOLE NO. 1
MANHOLE NO. 2
MANHOLE NO. 3
MANHOLE NO. 4
MANHOLE NO. 5
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36
TIME SCALE IN MINUTES.

FIG. 2.

per second for 6 ft. head. These conditions it was thought approximated closely enough to the workings of a flush-tank. A 5-in. opening was cut in a pine board firmly held against the end of the 8-in. pipe; then a flat cover, 6 ins. in diameter and faced with rubber, was provided, which, placed over the opening and held there by a light stick braced against the back of the manhole, made an effective plug. The manhole was filled to any desired depth by means of fire-hose attached to neighboring hydrants, and then, by means of a cord fastened to the stick and to the cover, the contents of the manhole were discharged into the sewer. The capacity of the manholes at depths varying by 6 ins. was determined by measurement, so that by filling to the proper depth any desired amount of water could be discharged. The effect of the flush waves was then noted at the

successive manholes down the line. No determinations of the velocity of the wave were made, the effect being judged by the depth of the wave, and by the force shown in moving gravel, etc., placed in the different manholes. The wave depths were read by different observers stationed in the manholes, where they recorded as rapidly as possible (usually every seven seconds) the depth as marked on a thin vertical scale placed in the sewer. Figs. 2 to 5 show the wave forms and the progressive flattening as the wave gets farther and farther from the flush-tank.

To test the transporting power of the wave small brickbats and gravel of various sizes, coated with paint so as to be recognizable, were placed in the inverts at the manholes. A considerable growth of what was apparently of vegetable origin had become attached to the sides and bottom of the pipe, and the value of the flush in removing this growth was also noted. The order of procedure was to examine and note the condition of the line, and, after placing the gravel, etc., to make a number of flushes, each of 20 cu. ft., and note the results. Then, increasing the amount discharged to 30, 40, 50 and 60 cu. ft., the respective results were noted. Then, either the whole pipe was scraped by a rubber-edged piston-like cleaner, or merely the manhole inverts and about 6 ft. each way into the pipe, and the flushing repeated. The following tables give the results on the different lines:

TABLE No. 2.—Green Street Sewer.

Volume of flush.	Effects at				No. of flushes.
	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	
25 cu. ft...	Scoured clean.	Scoured clean.	No effect.	No effect.	1
30 " ...	"	"	"	"	1
40 " ...	"	"	Several stones started.	"	8
60 " ...	"	"	Small gravel generally started.	"	2
80 " ...	"	"	"	"	2
120 " ...	"	"	"	"	3

Before commencing the work, the examination of the pipe showed it to be practically clean, with no ground-water, except between the third and fourth manholes, where there was a stream perhaps ½ in. deep. There were no house connections, but there was a small depth

of silt, and small pieces of cement left from construction, also a slight growth on the sides and bottom of the pipe. Gravel of all sizes placed in the pipe at the flush-tank was carried through to manhole No. 1 in two flushes of 25 cu. ft. each, the first flush alone not being sufficient. The gravel scoured out of the bottom of No. 1 manhole by the first flush was not brought to No. 2 until the 80-cu.-ft.

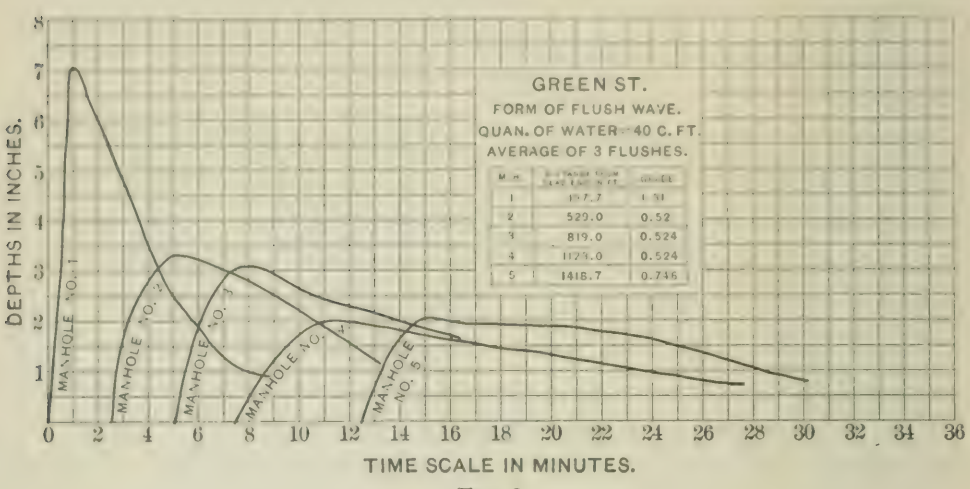

GREEN ST.
FORM OF FLUSH WAVE.
QUAN. OF WATER 40 C. FT.
AVERAGE OF 3 FLUSHES.
1 157.7 1.31
2 529.0 0.52
3 819.0 0.524
4 1178.0 0.524
5 1418.7 0.746
DEPTHS IN INCHES.
MANHOLE NO. 1
MANHOLE NO. 2
MANHOLE NO. 3
MANHOLE NO. 4
MANHOLE NO. 5
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36
TIME SCALE IN MINUTES.

FIG. 3.

flush was put in, and no gravel scoured out of No. 2 was brought to No. 3 by any of the flushes. After the seventeenth flush as above, the pipe was thoroughly scraped and cleaned, and flushes eighteen to twenty-eight made. Similar results were obtained, except that the flushes carried the gravel about 200 ft. farther than before and seemed effective for that distance.

TABLE No. 3.—CAYUGA STREET SEWER.

Volume of flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of flushes.
30 cu. ft...	Scoured clean.	No effect.	No effect.	No effect.	3
40 " ...	"	Disturbed but not cleaned.	"	"	7
60 " ...	"	Partly scoured.	Some vegetable growth passed through.	"	2
80 " ...	"	Cleaned.	"	"	3

In Cayuga Street there were a few connections and little flow, so that the condition of the pipe was very foul; there was also a heavy vegetable growth in the pipes.

On Linn Street no comparative records could be made. The pipe was clean from the flush-tank to Manhole No. 1, and in this length there were no connections. From No. 1 to No. 2 it was slightly foul, and very foul the remainder of the length. There were two house connections on the line. Five flushes of 20 to 60 cu. ft. were made.

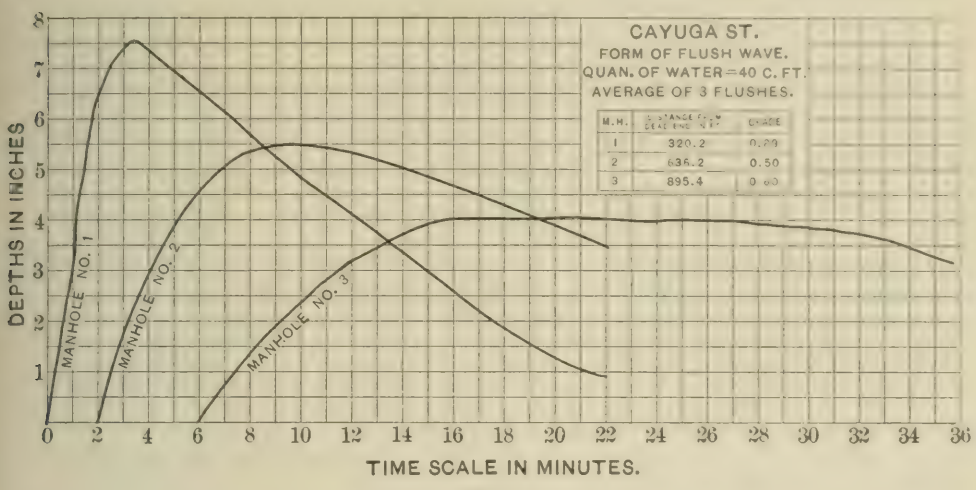


FIG. 4.

Each was very effective, one apparently as much so as another. All obstructions introduced were removed at once from manholes Nos. 1 and 2. A steady flow 1 in. deep from the hose carried everything forward at once to a point beyond No. 2 and to the flatter grade.

TABLE No. 4.—AURORA STREET SEWER.

Volume of flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu. ft....	Cleaned.	Cleaned.	No effect.	No effect.	3
60 "	"	"	Disturbed.	Water dirty; some vegetable growth came through.....	7
80 "	"	"	"	A few stones disturbed.	2

TABLE No. 5.—FIRST STREET SEWER.

Volume of flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu. ft....	Cleaned.	No effect.	No effect.	No effect.	5
60 "	"	"	"	"	3
80 "	"	"	"	"	2

On the Aurora Street line, the pipe was very foul, chiefly from a hospital connection at the upper end. The vegetable growth was large, and the accumulations of organic matter very evident.

On Buffalo Street, where the grade is about 12%, the effect of the flush was amazing. Where any sewage at all flows in the pipe, it is

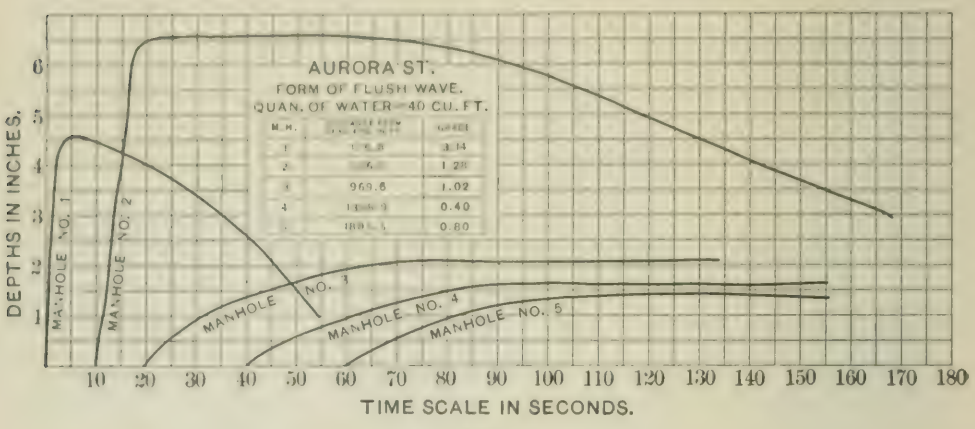


FIG. 5.

sufficient to remove all obstructions. A flush of any volume rushes down the hill at a high velocity, with piston-like action, and sweeps everything before it.

Table No. 6 gives the distances and grades between manholes on the lines used in the experiments.

TABLE No. 6.—DISTANCES AND SLOPES BETWEEN MANHOLES.

Description.	GREEN ST.		CAYUGA ST.		AURORA ST.		FIRST ST.		LINN ST.	
	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.
Dead end to Manhole No. 1	298	1.31	320	0.89	177	3.14	371	1.00	331	2.94
Manhole No. 1 to Manhole No. 2	231	0.52	316	0.50	390	1.28	341	0.50	278	2.70
Manhole No. 2 to Manhole No. 3	290	0.52	259	0.60	413	1.02	394	0.57	317	0.50
Manhole No. 3 to Manhole No. 4	305	0.52	.....	.....	419	0.40	393	1.00	.....	.....
Manhole No. 4 to Manhole No. 5	296	0.75	.....	.....	417	0.80	.....	.....	.....	.....

The manager of the Van Vranken Flush-Tank Company gives his practice in proportioning the sizes of flush-tanks for any particular

sewer, as follows: The capacity of the reservoir should be equal to one-half that of a length of sewer in which the grade produces a rise equal to the diameter of the pipe; so that the Green Street line, 8 ins. diameter, and 0.5% grade, should have a discharge of half the volume of the pipe, $\frac{4}{3} \times 100$ in length, or 23 cu. ft.; and for a 1% grade one-half of that, or 11.5 cu. ft. He says further, and the statement has been confirmed by the author's work, that an 8-in. pipe on a 0.4% grade will flow about one-third full at a distance of 300 to 400 ft. from the tank discharging the above amount; and that on a 5% grade the water will come down as a solid piston for any discharge greater than 14 cu. ft.

The manager of the Pacific Flush-Tank Company writes that as a rule he does not interfere with engineers in their design for tanks, but, in his opinion, a flush of 175 galls. on a 1% grade is sufficient, and on any flatter grade twice that amount of water should be used, or, in other words, as he says, "long lines or flat grades require greater capacity of tanks than steep grades or short lines."

Conclusions.—The following conclusions are based upon data on this subject published previously; upon the experience of engineers in different parts of the country; upon the flushing diagrams published recently by J. W. Adams, and upon observation and the special experiments made in Ithaca; and it is believed that they are justifiable and are a safe guide as to the use of flush-tanks.

(1) Flushing of some sort is required at the upper ends of laterals, the frequency and amount depending on the number of house connections, on the carefulness or prodigality in the use of water by the householder, on the grade and size of the sewer, on the character of its construction, and on a mysterious something which defies definition, but which produces frequent accumulations in one line and does not affect another, apparently like the first.

(2) This variety in the conditions prevents any exact statement of a relation between the quantity of water which should be discharged from a flush-tank and the grade of a sewer, but it plainly indicates that the advantage of automatic flush-tanks lies in a general guarantee or insurance against accumulations in the upper part of the laterals, while periodic hand-flushing must be depended on only when in charge of a responsible, indefatigable and intelligent caretaker.

(3) Judging by the experience at Ithaca, and despite the statements of other engineers, it seems to the author that on grades of less than 1%

automatic flush-tanks are an economic necessity, even where water has to be paid for, the added expense of frequent hand-flushing more than off-setting the possible discharge of flush-tanks when not absolutely necessary.

(4) The volume of water discharged should not be less than 40 cu. ft., and the effect of the flush can hardly be expected to reach more than 600 or 800 ft. Below this point accumulations may occur which must be removed by hand-flushing and carried on to a point where the sewage flow has the necessary transporting power.

(5) On flat lines and where obstructions occur below the influence of the flush-tank, a second flush-tank, placed about 800 ft. from the first, will be more effective than increasing the first tank to a capacity of three times its original discharge.

(6) The frequency of discharge should depend on the local conditions, but it is probable that the maximum interval depends on the practical working of the siphon, so that the usual prescription of once in twenty-four hours is a safe rule.

(7) If tanks are used on grades greater than 1%, 15 to 20 cu. ft. give as good results as larger amounts, with the same rule as to frequency of discharge.

(8) However, economy is best served, on grades above 1%, by omitting flush-tanks, and resorting to periodic hand-flushing at such intervals as experience shows to be necessary on the different lines. In most cases semi-annual or quarterly flushings, with a hose, are sufficient.

(9) On grades greater than 3% flush-tanks are unnecessary, and their installation is a waste of money.

(10) Hand-flushing should be performed and tanks discharged at night, as a flow of even an inch in a sewer offers a large resistance to the flushing action; while, with a pipe flowing half full, the effect of a flush-tank is scarcely visible.

DISCUSSION.

Mr. Hering.

RUDOLPH HERING, M. Am. Soc. C. E.—Words of encouragement should be given to every one who finds and utilizes an opportunity to undertake experiments on this subject, as there is but little useful information concerning it in existence.

The experiments described in the paper are valuable, so far as they go, and furnish data from which a better judgment, than was possible before, can be obtained on a number of points. The author confines his recorded experiments to 8-in. pipes, and to the same quantity of flushing water in each case, varying only the gradients of the pipes.

The answers received in response to the postal card inquiry indicate first, that "occasional flushing on low grades, probably below 1%, is needed at the upper ends of lateral sewers," and, secondly, that automatic flushing requires less labor and less vigilance than hand-flushing, though, without proper inspection, the former allows, as he says, "a serious nuisance to result." The speaker thinks that the automatic flush-tank should not in any measure be made responsible for "the serious nuisance," but that the responsibility, in such cases, is entirely due to the failure of sufficient or efficient inspection, a *sine qua non* for every sewerage system that is to be kept in order, whether automatic flush-tanks are used or not.

The author did not make his experiments with automatic tanks, but by filling a manhole with an amount of water and drawing a plug, by which the water escaped into the 8-in. sewer, through an opening 5 ins. in diameter cut in a pine board. This circumstance should be emphasized a little, so that the results may not be quoted as applying to automatic tanks. In the latter the rates of discharge and the velocity of the water issuing from the tank will differ, although not materially, from those of the former case.

It is also advisable, in such experiments, to have the velocity of the water measured at different parts of the wave, instead of calculated from the gradient or judged by the force of moving gravel, etc. It is to be hoped that such velocity measurements may be made at an early day, as a formula gives but a rough approximation to the velocity. In the same pipe with the same section and slope of the water, the velocity will be different when the water is rising from that which obtains when the water is falling. The usual slope formulas take no account of this fact. It has been reported that the stream flows quicker just after the crest of the wave has passed a manhole than before it arrives at this place. Further, the loss of head in the cleaning process, by loosening and carrying along deposit and sticky material would also be properly accounted for by actual velocity measurements, if such were made.

Mr Hering. The European experience with flushing sewers dates back almost fifty years. In the speaker's report on this subject in 1880,* the results of such experience are given. There has been almost no progress made since then in the methods used, but, fortunately, flushing has become more common. It was almost nowhere practiced in America before 1880.

The principle that "long lines of flat grades require greater quantities than short lines of steep grades" was then well recognized in Europe. Small tanks were used only at the heads of short or steep sewers, where flushing requires but a small quantity of water. For flushing large sewers on flat slopes, the sewage itself was stored for this purpose in large quantities, or ground-water was collected in large cisterns, because in these cases a large quantity of flushing water is required. It had also been stated in Europe that daily flushing with automatic tanks was not everywhere considered necessary. It was the practice then, as it is now, to flush the smaller sewers by hand at intervals varying from twice a week to twice a month.

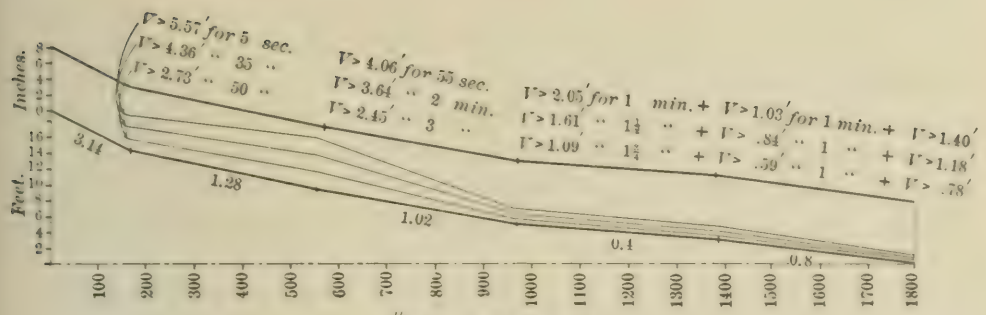
The assertion made by the author that automatic flush-tanks cause an additional expense of more than 8% of the cost of the system is hardly fair. He estimates $50 as the cost of such a tank, but apparently overlooks the necessity for having a manhole at the head of the sewer in any event, and he should charge the flush-tank only with the necessary appliances for flushing, or $15 to $20 instead of $50.

He also charges the flush-tanks with the annual expense for water and more intelligent maintenance. No doubt 20 cents per thousand gallons may be chargeable in some cities, and where private companies have a franchise granted on liberal terms; but the average cost of water in American cities is by no means so great. Its average value is much less, and in many cases is less than half of that figure.

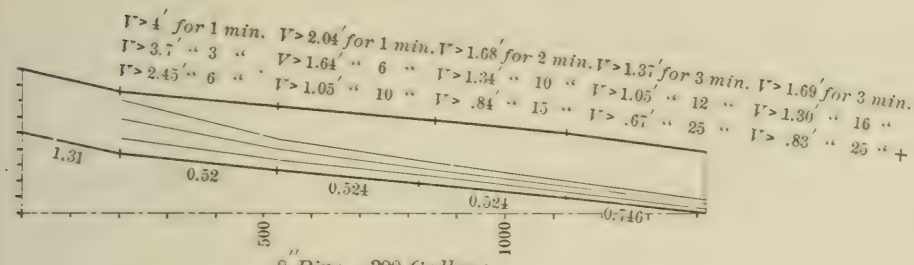
The author's deduction that the cost chargeable to flushing sewers amounts to about one-half of the construction account is, therefore, wide from the true mark, if it were generally applied. In the case of Ithaca, this conclusion should be modified, not only with reference to the cost of the tanks, but also with reference to the annual charge of $600, to examine and adjust 131 tanks. This cost is quite excessive when the best tanks are used.

In his opening paragraphs the author makes a rather sweeping assertion regarding the present extensive use of automatic flush-tanks for small pipe sewers. In some cities, particularly in those having adopted systems on the recommendation of Colonel George E. Waring, Jr., a very extensive use has been made of such tanks; but in most others their use has been limited to cases where they were more economical than hand-flushing, which fact needs determining in each case.

* *Transactions*, Vol. x, p. 361.



8″ Pipe. 300 Gallons



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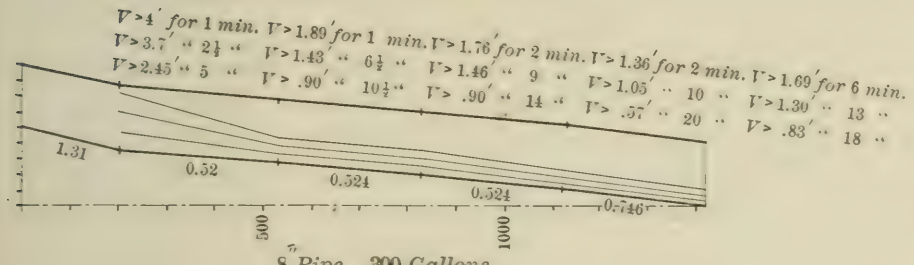
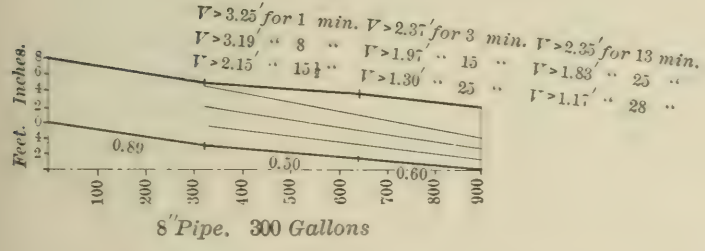


8″ Pipe. 300 Gallons



8″ Pipe. 300 Gallons

Volume of Flush given in U.S. Gallons. Velocities given in feet per second.
Horizontal scale gives distances in feet.
Slopes of sewer bottoms given in feet per hundred.
Velocities are calculated (Kutter, n=0·013), and are those due to the depths of flow at the respective points, and to the slopes of the sewers.

FIG. 6.

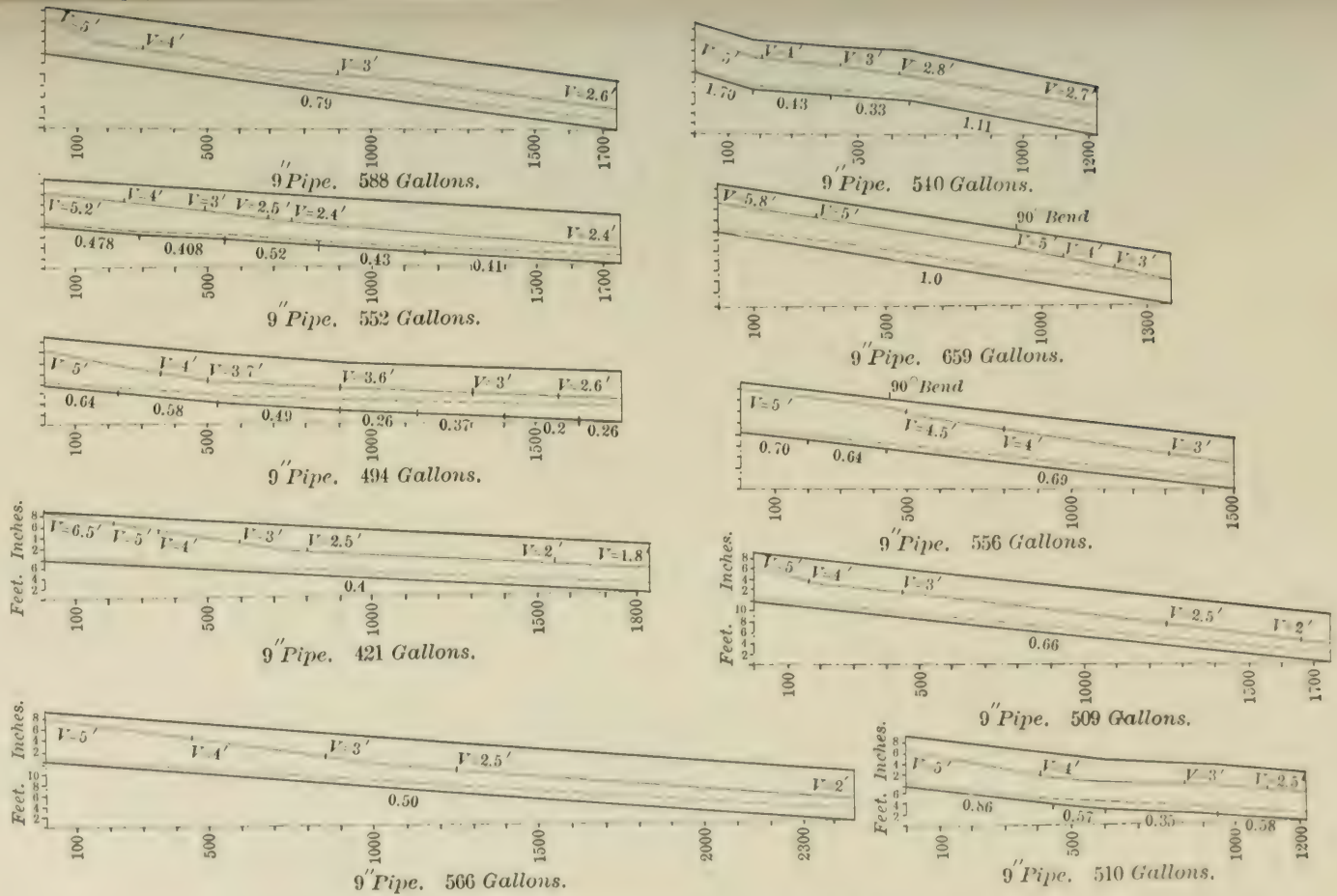
Mr. Hering. The author intimates that the suspending and scouring power of water depends on the velocity and also on the depth of the stream. The depth, of itself, irrespective of the velocity, has nothing to do with this power. Suspension in running water, and, even more so, the scouring power, are almost wholly functions of the velocity. In two streams having different depths, but equal velocities, the suspending and scouring power of the water will also be about the same. In two streams of water having equal depths, but different velocities, the suspending and scouring power will again depend practically on the velocity alone.

The material which is taken out of a sewer, when it is flushed quarterly or semi-annually, is sometimes a growth, a mycelium—the fungus growth in the sewer. It is, perhaps, not offensive, but it obstructs the flow and makes necessary the flushing of the sewer to obtain the full capacity again. At Colorado Springs, Colo., this fungus grows at such a rate that if the sewers were not cleaned out often, they would be completely closed up. The speaker was present when flushing was going on, and was astonished to see the large mass of fungus, amounting to several cubic yards, which came out of the sewer within a few minutes.

Referring to the conclusions at the end of the paper, the speaker is at a loss to understand the "mysterious something which defies definition," but which is said to produce frequent accumulations in one line of sewers and not in another. Such a mystery has never seemed to arise in the speaker's experience, and if the cases are sufficiently analyzed, he doubts the existence thereof. If, on one hand, the nature of the sewage discharged is observed, and, on the other hand, if the actual velocity measurements are made, as suggested above, any mystery would probably disappear.

Another conclusion states that "a flow of even an inch in a sewer offers a large resistance to the flushing action." This statement in its implication can certainly not be correct. Sewage is but dirty water, and whether the first inch of flow is sewage or clean water should make no practical difference in the velocity of the flushing water. Certainly the flow of sewage would not offer a large resistance when the flow of clean water offers but a slight resistance. The deposits to be overcome are, of course, supposed to be the same in both cases. Therefore, the speaker cannot agree with the author's proposition that flushing should be done at night. On the contrary, to give the best effect, it should be done when the flow of sewage is greatest, that is, in the forenoon. The flushing water, when added to the sewage, will increase its flow, and, therefore, also its velocity, its scouring and its cleansing effect, to the greatest extent.

The author's diagrams, representing the results of some of his experiments, are very interesting and valuable. They are plotted so that



Volume of Flush given in U.S. Gallons. Velocities given in feet per second.
Dotted line shows depth due to sewage. Horizontal scale gives distances in feet. Full line shows depth due to flush.
Slopes of sewer bottoms given in feet per hundred.

FIG. 7.

Mr. Hering. the axis of abscissas is the time scale, and the ordinates represent the depth of the water.

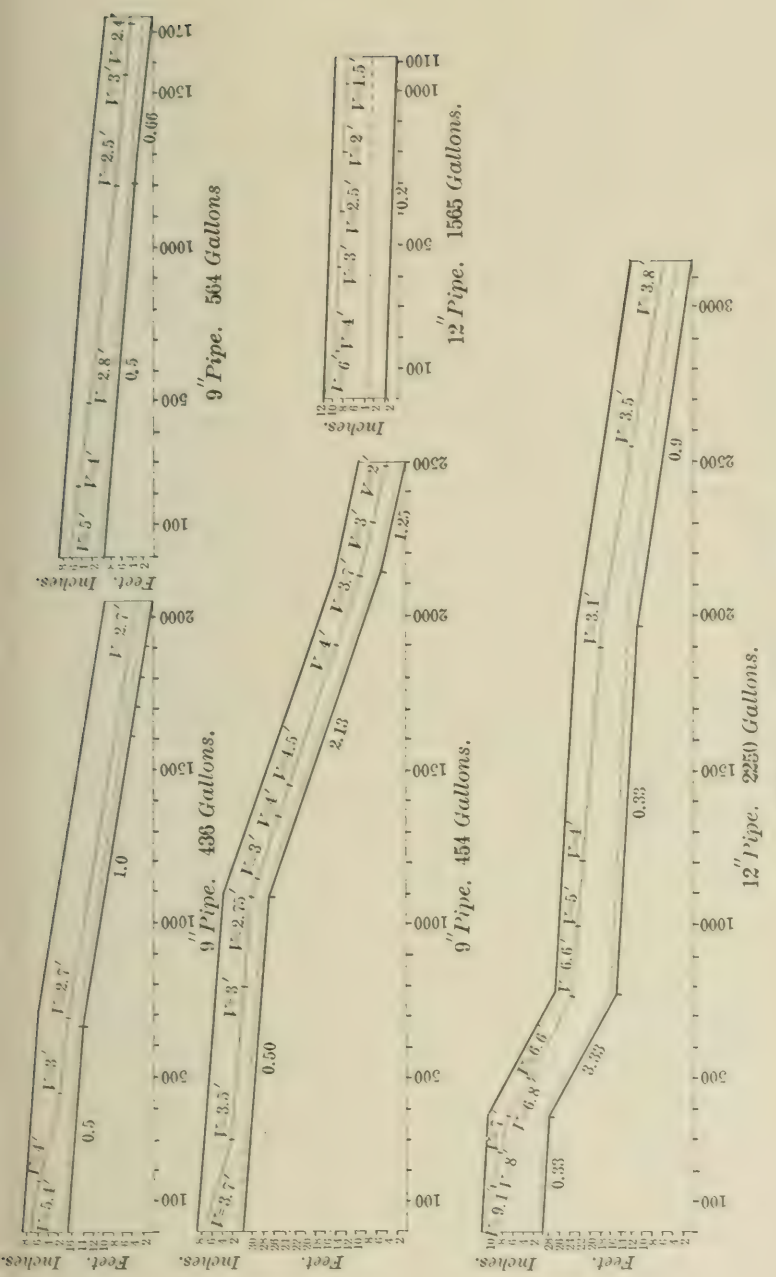
Last year some diagrams were published,* which likewise show the effects of sewer flushing. These are plotted so that the abscissas represent the distance traveled by the flush wave, and the ordinates the velocity of the water. To enable one to further appreciate these results, another plotting is possible, partly combining these two methods and letting the abscissas represent the distance or length of flush and the ordinates the depth of the water. The advantage of this plotting is the possession of a true profile of the sewer and of the flush wave, in which one can see its length and depth at a glance, while the velocities are written down in figures above. The Ithaca data are thus plotted in Fig. 6, and the Adams' data in Figs. 7, 8 and 9. James H. Fuertes, M. Am. Soc. C. E., has kindly assisted the speaker in replotting the above-mentioned diagrams in this manner.

The Ithaca data (Fig. 6), thus re-plotted, show how the velocity gradually decreases with the distance, and show the distances and depths of flow up to which the necessary cleansing velocity is maintained. They also show by figures how long such velocities continue. As stated above, these velocities were not measured, but merely computed. They would, therefore, be somewhat different in practice, but how much is not yet known. As an example, when the flush wave is highest, a velocity of more than 2 ft. per second is maintained for only one minute, and it extends less than 500 ft.; when the water flows at $\frac{2}{3}$ this height, the velocity has continued for over five minutes, and for this length of time it extends only about 400 ft.; and when the water flows at $\frac{1}{3}$ its greatest height, a velocity of over 2 ft. per second has continued for about 7 minutes, but for this length of time has extended only about 300 ft.

This diagram clearly shows the well-known fact, that the greatest benefit from flushing is obtained by maintaining the cleansing velocity as long in time and as far in distance as possible. It is, therefore, desirable to determine for each case approximately the quantity of water needed to obtain a required and definite result in a sewer of a given size or slope, and the author's work materially facilitates this determination for the cases which he has covered.

Mr. Fuertes. JAMES H. FUERTES, M. Am. Soc. C. E.—The author has rendered the profession a valuable service in recording the results of his experimental flushing of sewers, but the speaker regrets that more data, covering a wider range of conditions, were not incorporated in the paper, and would respectfully suggest that the records would have been more complete had he given the elevations of the surface of the water in the flush-tanks as well as in the manholes. The pages of the *Transactions* of this Society are the proper repositories for as many

* By Messrs. Adams & Co., Engineers, of London, England.



Volume of Flush given in U.S. Gallons. Velocities given in feet per second.
Dotted line shows depth due to sewage. Horizontal scale gives distances in feet. Full line shows depth due to flush.
Slopes of sewer bottoms given in feet per hundred.

FIG. 8.

Mr Fuertes. such records as can be obtained, and it is to be hoped that in closing this discussion the author will offer as much matter as is practicable.

The data presented by the author permit the drawing of some interesting inferences concerning the changes, as to form of wave and velocity of flow, taking place in a quantity of water discharged suddenly, under a head, into a sewer; and permit the correction, by graphical demonstration, of some popular and fallacious notions concerning the action of a flushing wave in a sewer. The idea is commonly expressed, even to-day, that, to be effective as a flush, the water must shoot through the sewer in a solid body like a piston. That this is not true will be demonstrated presently.

In conclusion 10 the author makes the following unqualified statement: "while with a pipe flowing half full, the effect of a flush-tank is scarcely visible." By this very broad assertion he creates the impression that no matter how large the flush-tank, or under how great a head it is discharged, the pipe would not run over half full. Surely this is not the meaning intended to be conveyed.

The speaker's hopes were raised to a high pitch, in reading this paper, when the author proposed the consideration of the hydraulic problems involved in flushing; but a careful perusal of the pages which followed failed to reveal the promised consideration of the subject.

Sewers are flushed with water to clean and wash them out, and to prevent their becoming foul. When deposits have taken place in sewers, due to insufficient grades, poor workmanship or other causes, two operations are necessary in order to remove these deposits. These are, first, to loosen and dislodge the deposits; second, to carry away, in suspension in the water, the matter thus dislodged. The question is: Do the most effective velocities for these two operations exist at the same moment at any given point, or, if not, when does each reach its maximum value ?

The scouring power of the wave will be greatest when the velocity at the bottom of the pipe is at the maximum. The transporting power of the wave, on a given grade, will be greatest at a certain point when the mean velocity at that point is greatest. The conditions for maximum scouring power and maximum transporting power are, therefore, not coexistent, because the maximum scouring power is at the toe of the wave as it descends the sewer, while the maximum transporting power occurs at a later time. On the diagram, Fig. 10, is shown the form of the flushing wave discharged into the Green Street sewer, drawn from the author's data. A profile of the wave is shown at the end of the 1st, 2d, 3d, 4th, 6th, 8th and 10th minutes. In this diagram the shapes of the waves are not claimed to be exact as to profile at all points. They are accurate at the points where they cross the

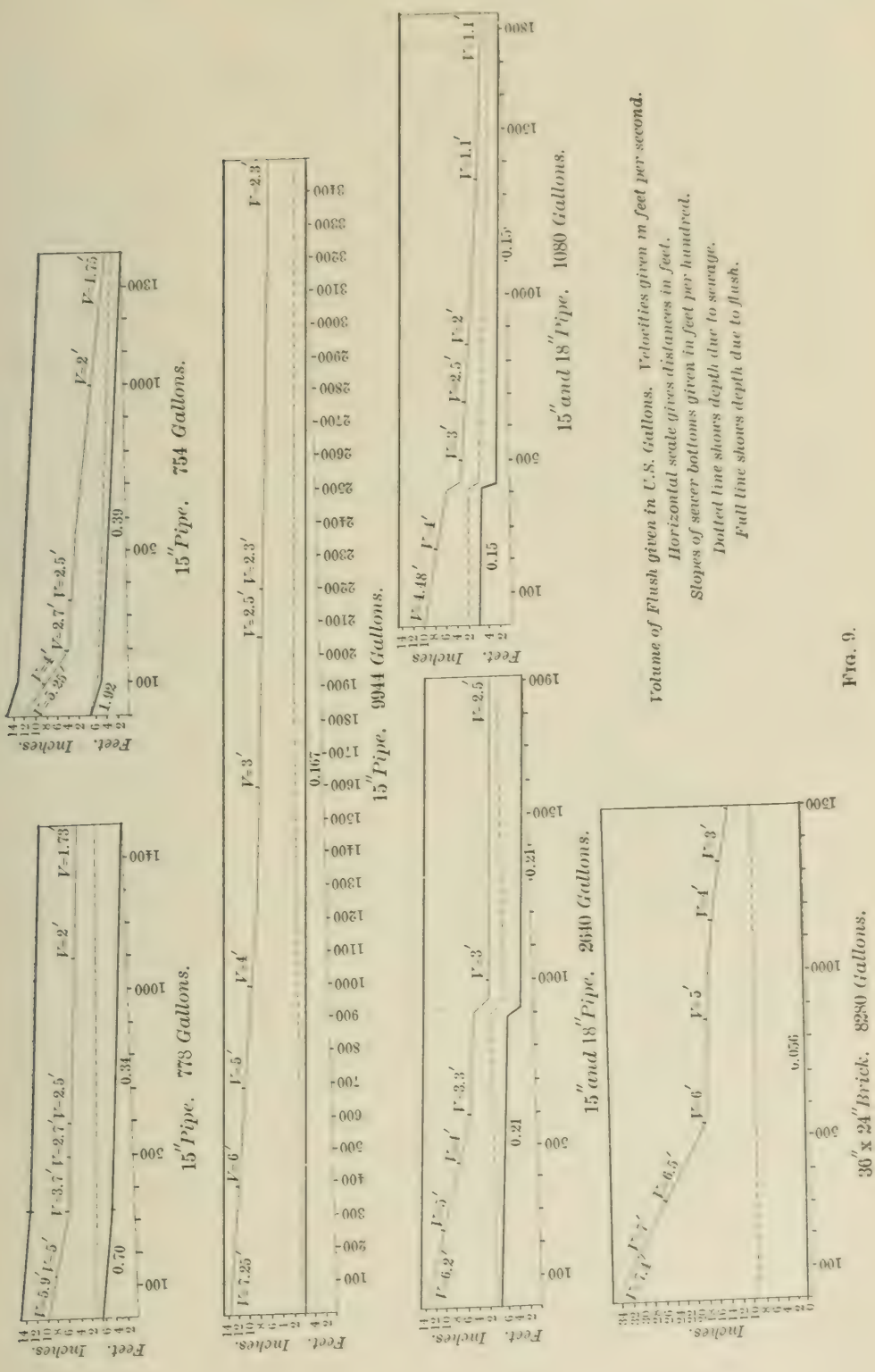

15″ Pipe. 754 Gallons.
15″ Pipe. 778 Gallons.
15″ Pipe. 9944 Gallons.
15″ and 18″ Pipe. 1080 Gallons.
15″ and 18″ Pipe. 2640 Gallons.
36″ x 24″ Brick. 8280 Gallons.
Volume of Flush given in U.S. Gallons. Velocities given in feet per second.
Horizontal scale gives distances in feet.
Slopes of sewer bottoms given in feet per hundred.
Dotted line shows depth due to sewage.
Full line shows depth due to flush.
Feet. Inches.

Fig. 9.

Mr. Fuertes. manholes, and the positions of the toes of the waves were interpolated; the remaining portions of the curves are sketched in. The velocities corresponding to the greatest depths of flow were calculated by the Kutter formula ($n = 0.013$). The velocities marked for the toe of the wave are the components, along the bottom of the sewer, of the velocities of the surface of the water at the given points when the wave reaches those points.

At the first manhole the sewer was nearly full, and probably had been running under a head up to within about 60 ft. of the manhole. After passing this point the foot of the descending body of water, under a free flow by gravity, rushed forward rapidly, the point being 135 ft., 265 ft., 391 ft., 513 ft., 630 ft., 743 ft., 852 ft., and 957 ft., distant from the manhole in 1, 2, 3, 4, 5, 6, 7 and 8 minutes, respectively. Traversing these distances in the times given corresponds to an average velocity between the respective points of 2.25 ft., 2.18 ft., 2.10 ft., 2.03 ft., 1.95 ft., 1.89 ft., 1.81 ft. and 1.75 ft. per second. These are the bottom velocities at the point of the wave, and are the actual maximum velocities at the respective points. When the greatest depth of flow has been reached, the slope of the water surface will be about the same as the slope of the sewer, and the mean velocity less (consequently, the bottom velocity will be much less) than at the point of the wave. Therefore, after the natural flow ensues, the maximum scouring power occurs at the point of the wave, and the maximum transporting power at the moment of greatest mean velocity at any section. This scouring power expends itself in friction and work, and very little head is left for velocity, consequently it has small transporting ability.

At any point the true flushing velocity, with a given volume of flushing water, is, therefore, that due to the grade of the sewer at the time when the depth of flow gives the greatest mean velocity. With deposits loosened up so that they may be entrained in the current, the best flush, in a sewer with a circular cross-section, would be that which would cause the greatest length of sewer to flow about eight-tenths full for a considerable length of time. These remarks apply, of course, to the sewer below the point where it runs full under a head. Under ordinary conditions, 8-in. sewers of moderate slopes will not generally run under a head from a flush-tank for more than 100 ft., and even in extreme cases not much over this limit. Of course, with flush-tanks which will discharge at a greater rate than the capacity of the sewer, on steep grades, the sewer may run with full section for a considerable distance; however, this condition is not frequently met in practice. Therefore, the natural flow in the sewer must be relied on to do the cleansing. In this discussion the term natural flow is used to indicate the flow due to the slope of the sewer only, not to any extraneous head from the flush-tank. A steady flow of water of consid-

erable depth for a considerable time, repeated at certain intervals, would, under most conditions, therefore, be of greater benefit than the frequent discharge of small flushes at the upper end. When a small flush is discharged from a tank the maximum depth of flow at different points will diminish as the wave descends the pipe, and, consequently, the farther away from the tank the less the transporting power of the wave. Particles that were picked up or rolled along by the current at certain points would be deposited again at points further down, and the power of the flush from a tank to remove these deposits will cease beyond a certain distance. Mr. Fuertes.

The matter which is most difficult to dislodge from the bottom of sewers is the deposit of street dirt and silt which becomes coated with grease and slime. The discharge from a flush-tank is of great value

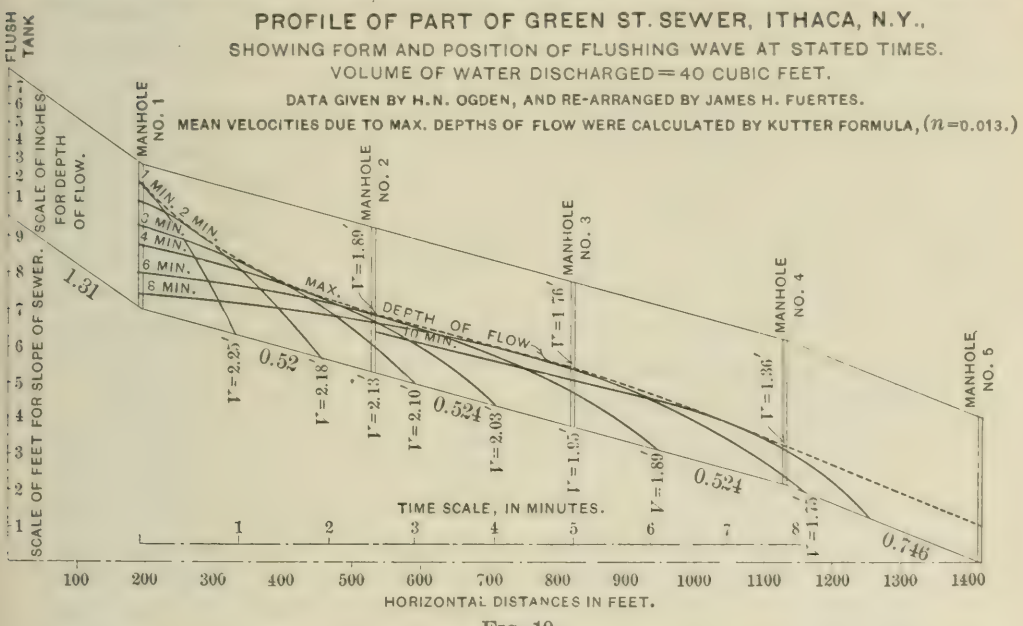

PROFILE OF PART OF GREEN ST. SEWER, ITHACA, N.Y.,
SHOWING FORM AND POSITION OF FLUSHING WAVE AT STATED TIMES.
VOLUME OF WATER DISCHARGED = 40 CUBIC FEET.
DATA GIVEN BY H.N. OGDEN, AND RE-ARRANGED BY JAMES H. FUERTES.
MEAN VELOCITIES DUE TO MAX. DEPTHS OF FLOW WERE CALCULATED BY KUTTER FORMULA, (n=0.013.)
FLUSH TANK
MANHOLE NO. 1
MANHOLE NO. 2
MANHOLE NO. 3
MANHOLE NO. 4
MANHOLE NO. 5
SCALE OF INCHES FOR DEPTH OF FLOW.
SCALE OF FEET FOR SLOPE OF SEWER.
2 MIN.
4 MIN.
6 MIN.
8 MIN.
MAX. DEPTH OF FLOW
10 MIN.
1.31
0.52
0.524
0.524
0.746
TIME SCALE, IN MINUTES.
HORIZONTAL DISTANCES IN FEET.

FIG. 10.

in loosening this up so that it may be carried along in the deeper water which follows the toe of the wave.

This graphical illustration of the form of the flushing wave dispels the illusion that to be effective the flush must go down like a piston. That the best results follow the use of a deep flow of water for a considerable length of time, has long been understood and recognized in England and Germany. Numerous cities could be quoted where the provisions for flushing have been designed on this basis. At Munich and Frankfort, for instance, very large underground reservoirs are built at the heads of several sewers, and provisions are made for diverting the water into different branches and creating a deep flow for a considerable time. In this system of operation the idea of a plug of water does not enter in any way ; the whole cleansing effect being de-

Mr. Fuertes. pendent upon the flow of water of considerable depth. It is not always possible, however, to obtain topographical conditions favorable to this method of flushing, and then it is necessary to resort to the expedient of intermittent and frequent flushings with comparatively small quantities of water. This may be done either by automatic flush-tanks, a hose or by a water cart. The method to be adopted will be the outgrowth of local conditions.

The speaker is inclined, in most cases, to favor the use of automatic flush-tanks for all dead ends of pipe sewers on all grades. The necessity for flushing exists on steep as well as on flat grades, for the purpose, not only of removing actual obstructions, but for preventing the growths of fungi in the pipes, for preventing the formation of obstructions and for aiding in the ventilation of the sewer. The reasons that he favors the use of tanks are: the potential guarantee against trouble, as stated above, and the cheapness of the method, as compared with other systems that are equally safe.

The author makes out a bad case against automatic flush-tanks, and intimates that those on the Ithaca sewers represent an expenditure equal to about one-half the cost of the system of sewers. In the first place it is stated that the cost of the flush-tanks should be estimated at $50 each. This is between two and three times the amount they would actually cost; as the manhole at the end of the sewer would be a necessity in any event, and as the amount chargeable to the flush-tank should be the cost of the apparatus and the cost of the labor for placing it in position.

He estimates the cost of the tanks at about 8% of the cost of the system, and adds that their use increases the cost of the separate system by nearly one-tenth. This argument against the use of tanks could be applied with equal force against the use of manholes, which are provided principally in order to locate obstructions should they occur, and the cost of which constitutes about one-fifth of the total cost of an ordinary system of pipe sewers. He estimates the total cost of water at $3.93 per day, or about $1 434 per year for flushing the Ithaca sewers. This amounts to about $0.15 per mile per day. If these sewers were to be flushed entirely with accumulated sewage by means of hand gates, without the use of city water, one man would have to keep clean about 10 miles of sewer per day in order to have the cost of flushing as little as the cost of the water (at Ithaca prices) to be used in the flush-tanks. The cost of the necessary hand gates, special constructions and apparatus would not be much less than the cost of the siphons for the automatic tanks; the repairs would not be much less, and the cleansing received by this method of flushing would not be as satisfactory in pipe sewers as would result from the use of flush-tanks discharging clean water into them.

Mr. Fuertes.

The author capitalizes the annual expenses chargeable to flush-tanks at about $40 000 and compares this with the actual cost of pipe laid. The comparison is meaningless, because the costs of water and maintenance are charged to the flush-tanks. These items should be charged against the sewers, as the expenditure is entirely for the purpose of keeping the sewers clean, and the only annual charge against the flush-tanks should be the interest and sinking fund charges on their cost and the actual labor necessary for their maintenance.

As to the necessity of using flush-tanks on 6-in. and 8-in. sewers on steep grades, it is possible, as the author states, that infrequent hand flushing may be quite satisfactory if properly attended to. As in most towns the number of such sewers is generally quite small, the omission of flush-tanks would probably cause a greater expense for maintenance than if they were provided.

With the author's ninth conclusion, that "on grades greater than 3% flush-tanks are unnecessary and their installation is a waste of money," the speaker is not in accord. Cases where such steep grades occur are comparatively rare, and no great mistake will be made by using flush-tanks at their upper ends, and if there is any mistake it will be on the safe side. It must not be lost sight of that these very heavy grades occur nearly always at the upper ends of the long sewers, and that the current of air through them will generally, though not always, be toward the top, at which point disagreeable odors will be noticed if the sewers are not kept clean.

The speaker would suggest, in closing, that with the great hydraulic laboratory recently instituted at Ithaca by the College of Civil Engineering of Cornell University, with which the author is connected, the opportunity exists of producing a great amount of valuable experimental data, and he hopes that the profession may soon feel the benefit of the work that may be done there.

Mr. Tillson.

G. W. Tillson, M. Am. Soc. C. E.—In Omaha the speaker once had an opportunity to see a good deal of the fungus growth alluded to by Mr. Hering. When the Omaha separate system was first constructed, a main sewer was built along the bluffs, and small 6-in. sewers were built at right angles thereto and running up through the business parts of the city. It was intended to have flush-tanks at the heads of these lines, but, because of some trouble over patents, some of the flush-tanks were not efficient. The grades at the lower ends of the sewers were about 6 ins. per 100 ft., while at the upper ends they were from 5 to 8 ft. per 100 ft. A year or so after the sewers had been constructed they became stopped, and upon examination were found to be nearly half full of this fungus growth for some hundreds of feet. Having been built without manholes, it became necessary to dig down and open up the sewers to clean them out, and, as it was found to be impossible to flush them out with water, they had to be cleaned out

Mr. Tillson. mechanically. The sewers gave this trouble as long as they were in use, and finally they had to be enlarged. As the system was extended, and flush-tanks, which flushed every twelve hours, were built at the upper ends of the lines, all such difficulty was avoided, and, with the exception of the first four or five lines that were built, there was no trouble at all from the fungus growth.

While at Omaha an opportunity was afforded to witness the efficiency of a good flush, although it did not come from a flush-tank. The speaker had charge of the construction of an 8-in. sewer, which emptied into a 12-in. sewer. The trench had been back-filled for about 300 ft. and a manhole constructed at the upper end. A storm arose, and, as the trench had not been entirely filled, the manhole was badly washed, and after the storm a large hole was found in it. Whether it had been broken in by some miscreant was never found out, but the manhole was found half full of mud, and it was presumed that the sewer was in about the same condition. Half way down to the outlet, about 150 ft., a T had been constructed in the sewer, and upon digging down at that point the sewer was found to be full of mud. The contractor looked decidedly blue, thinking it would be necessary to take up the entire sewer, clean out and relay it.

The soil in the Missouri valley, or in the Mississippi, as is well known, becomes very slippery as soon as it is wet. Thinking to take advantage of this fact, a fire hose was attached to a hydrant and applied to the T. After the pressure, which at that point was about 90 lbs., had been applied for several minutes, the mud commenced to start, and in a short time it was driven out of the pipe for 150 ft. The hose was then applied to the manhole, and, after some delay, the sewer for the whole length of 300 ft. was cleaned out simply by flushing; the dirt coming out at the lower end in the form of a sausage. Of course, this was only possible because of the nature of the soil, which, as above stated, is very slippery, and when wet is almost like grease.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E.—In conclusion No. 8, there is a reference to semi-annual or quarterly flushings with hose, which are sometimes sufficient. The speaker would like to know what kind of matter is swept out of a sewer by those flushings, or, particularly, whether it is sediment from the sewage itself or sand that has accumulated? It is important to keep a sewer reasonably clean. If sewage remained, it would become nearly as foul and objectionable in a fraction of one month as it would in any portion of a year. If the flushing is simply to free the sewer from sand and restore its former capacity, the interval would depend upon many external conditions, and could hardly be fixed by a general rule.

CORRESPONDENCE.

Mr. Landreth.

WM. B. LANDRETH, M. Am. Soc. C. E.—This paper is of value in that it gives the results of observations of the form and intensity of the flush wave in small sewers and conclusions based on the author's personal experience and correspondence with other engineers.

The writer has had occasion to use flush-tanks on several sewerage systems, both separate and combined, and has closely watched their operation, to determine, if possible, the relations between the grade of the sewers, the amount of water necessary for flushing and the frequency of its application, but he has been unable to arrive at any exact results. The condition of the sewers, whether clean or foul, smooth or rough, has a great bearing upon the amount and rate of flushing necessary. The rate of discharge from the flush-tanks also has some influence upon the results, and, in the writer's opinion, should not be less than from 10 to 15 galls. per second for 8 or 10-in. sewers on grades less than 1 per cent.

The author's conclusion that flush tanks may be profitably used at intervals along a sewer seems correct, especially on flat lines at the foot of steep sewers, as the checking of the sewage flow when it reaches the flat grade causes it to drop the sediment, which has been easily carried on the steep one.

The economic regulation of the flush-tanks on a sewerage system is not easy, and can only be ascertained by experiment, as the quantity of water necessary will vary on different lines.

The conclusions reached by the author seem to the writer to be conservative, and to agree closely with the results of his own observations.

Mr. Le Conte.

L. J. LE CONTE, M. Am. Soc. C. E.—The necessity of flushing the upper ends of laterals is generally conceded to be a good sanitary measure. The three main principles to keep in mind are efficiency, economy and elasticity. By elasticity is meant easy adaptability to meet all natural changes in the conditions of flow in the sewers of a town, resulting from an increase in population. Experience teaches the writer that the use of fire hose or a direct connection with the water mains of the city fulfills these three conditions to the best advantage, all things considered.

Flush-tanks are expensive and require more or less attention all the time, in order to maintain the flushes at suitable intervals. To avoid vexatious interruptions, the common practice is to increase the supply of water to each tank, thus shortening the intervals between flushes. Where a city is largely supplied with flush-tanks, the willful waste of water due directly to this cause is something almost incredible. The writer knows of one or two extreme cases where towns

Mr. Le Conte use flush-tanks on a large scale, and nearly half of the entire daily water supply of the town passes through the flush-tanks and into the sewer system. This is always the case where the town or city owns the water supply, or where water is furnished free for all public purposes.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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No. 834.

THREE-HINGED MASONRY ARCHES; LONG SPANS ESPECIALLY CONSIDERED.

BY DAVID A. MOLITOR, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

WITH DISCUSSION.

INTRODUCTORY.

The advantages which well-designed masonry arches offer, as compared with the less durable structures of iron and steel, have been adequately demonstrated by modern experience.

The cost of maintenance of iron and steel bridges, together with their more or less limited lasting qualities, are sometimes offset by the ease, simplicity and accuracy of design and erection to which they are susceptible. The time allowable for construction may also, in many cases, weigh strongly in their favor.

However, the many masonry arches built centuries ago—a few antedating written history—are indisputable evidences of permanency. Few of these arches have required any repairs, and their cost of maintenance has amounted to almost nothing, a fact not to be realized in metal bridges.

The purpose of this paper is to demonstrate that masonry arches may be constructed on any good foundation, such as hard clay, and that they will admit of an accuracy and simplicity of design quite equal to that attainable for similar structures of iron or steel. In many instances concrete arches are even cheaper in first cost than metal bridges. Besides, the former possess the additional advantages of permanency and low cost of maintenance.

Recent progress, achieved through the earnest labors of German, French and Austrian engineers, has destined the masonry arch to become the successful competitor of iron and steel bridges, whenever the natural conditions of foundations and length of span do not offer unsurmountable difficulties.

The great advances accomplished in the manufacture of cements during the past ten years, the elaborate arch tests made by the Austrian Society of Engineers and Architects from 1890 to 1895, and the construction of a few three-hinged masonry and concrete arches, venturing the adoption of high unit stresses, low factors of safety and long spans; these mark the arrival of a new era in masonry bridge construction.

However, many difficulties are encountered in the construction of fixed masonry arches, owing particularly to insufficient elasticity in the masonry. The natural deformations in the arch, caused by shrinkage of the masonry, setting of mortar, stress and temperature, usually cause cracks which, while rarely of a serious character, are reasons for discouragement to the engineer, who has probably applied every known precaution to prevent their occurrence.

According to the recommendations of the Austrian Society of Engineers and Architects, as a result of their elaborate tests, a fixed masonry arch should be constructed only when the following conditions can be realized:

1. The abutments must be absolutely rigid.

2. The falsework must retain its form during the construction of the arch ring.

3. The material (stone and mortar) must be of the best quality.

4. The construction of the arch ring must be most carefully conducted.

5. The falsework must not be released until the mortar has thoroughly set.

6. When the falsework is released it must be done gradually and uniformly.

These conditions, except the two first named, can always be fulfilled, though the lack of rigidity of abutments and falsework are the two great obstacles in the way of long-span masonry arches without hinges.

In matters pertaining to the design of fixed masonry arches, it is safe to say that the method based on the theory of elasticity is the only one entitled to full confidence, and permitting of an analysis corresponding in accuracy with the knowable properties of the material. All other methods are too approximate to admit of close designing, such as the modern status of engineering science would generally demand.

This modern and most exact method, however, is not free from criticism. While the fundamental principles of the theory are almost axiomatic, their final application to the solution of stresses is extremely complicated, so much so that few engineers can be credited with the patience and earnest endurance to master either the method or the solution of a problem to which it is applied.

Therefore, unless the masonry arch can be so treated as to combine clearness, simplicity, undoubted accuracy and economy in design with faultless construction, the field of usefulness of this class of structure will remain restricted, and such monuments as the Cabin John Bridge will continue to remain curiosities of rare production.

This is not what the masonry arch deserves, in view of its practically everlasting life, nominal cost of maintenance and naturally æsthetic form, which latter should be a prime factor, though rarely given much consideration, in the choice of a bridge.

Essentially all the harassing features of fixed masonry arches are overcome by the introduction of hinges at the crown and abutments, thus permitting a rigid analytical treatment and affording almost absolute safety against cracks, even though small settlements may take place in the abutments. The idea was introduced by Koepke, of Dresden, in 1880, by providing open joints at crown and haunches. Karl v. Leibbrand, Stuttgart, in 1885, substituted sheet lead for the open joints, and in 1893, applied cast-iron, hinged bearings. The author, as early as 1888, while engaged on the construction of the strategical railway Weizen-Immendingen, Baden, Germany, advocated metal hinges for masonry arches, but prejudice and custom prevented a practical application being made at that time.

Some of the noteworthy bridges which have been constructed with hinges or hinge-like joints are briefly described in the following:

1. Bridges on the railroads of Saxony, built in 1880, by Koepke. The largest was of sandstone, 13 m. span, 3 m. rise, thickness of arch ring 0.50 m. to 0.60 m. Hinges consisted of a convex surface of sandstone, radius = 0.977 m., rolling in a concave surface, radius = 1.105 m. The maximum unit stress was 12.87 atm. Several arches of this type were constructed, some with only two hinges and some of concrete. All gave excellent satisfaction.

2. Sandstone bridge over the Enz River near Hoefen, Germany, built by Leibbrand, in 1885. Span, 28 m.; rise, 2.8 m.; maximum stress, 24 atm. Hinge-like joints of sheet lead. Several other bridges of this type were built in 1886 to 1890. The unit stresses were successively increased until 56.4 atm. were attained on the Forbach Bridge, in Baiersbronn, using sandstone of 653 atm. breaking strength.

3. Concrete arch over the Danube River, near Munderkingen, Wurtemberg, built by K. v. Leibbrand, in 1893. Span, 50 m.; rise, 5 m.; thickness of arch, 1 m. at crown, 1.4 m. at quarter points, and 1.1 m. at abutments. This arch was constructed as a three-hinged arch, and was the first masonry arch with actual hinged joints. The maximum compression in the arch was 34.6 atm. and 57 atm. adjacent to the steel hinges. The concrete was composed of 1 part Portland cement to $2\frac{1}{2}$ parts sand and 5 parts broken limestone, showing an ultimate compressive strength of 254 atm. in 28 days and 520 atm. in 2 years and 7 months. The settlement at the crown, from the time of closing the arch ring to the entire completion, was 13.1 cm. One abutment is founded on rock, the other has a pile foundation.

4. Concrete bridge over the Danube near Rechtenstein, Wurtemberg, built in 1893, by Engineer Braun. This bridge is made up of two arches, each of 23 m. span and 2.5 m. rise; the thickness of the arch is 0.65 m. at the crown and 0.9 m. at the haunches. Concrete for arch ring was composed of 1 part Portland cement to $2\frac{1}{2}$ parts sand to 5 parts gravel and $\frac{1}{8}$ part quarry stone. The hinges are 18-cm. and 20-cm. lead strips for the crown and abutments, respectively. The highest stress in the arch ring was 18 atm., and the settlements at the crowns of the two spans were 4.0 and 3.0 cm. One abutment was founded on piles, the other on gravel and boulder strata, and the middle pier on solid rock.

5. Bridge de la Coulouvrenière, over the Rhône River, in Geneva, Switzerland, built in 1895, by Engineer Butticaz. This bridge was made of concrete and consists of two main arches spanning 40 m. each, with a rise of 5.55 m. each, separated by a small span of 14 m., and a 12-m. arch adjoining the abutment of one of the main arches. The large spans were patterned after the arch at Munderkingen, and the small spans were supplied with lead joints, as the bridge at Rechtenstein.

6. Concrete bridge over the Danube, near Inzighofen, Wurtemberg, built in 1896, by Max Leibbrand. Span, 43 m.; rise, 4.46 m. hinged

at crown and abutments with cast-iron, hinged pedestals. Thickness of arch ring, 0.7 m. at crown, 1.1 m. at quarter points and 0.78 m. at abutments. Maximum stress, 36.5 atm. in compression, and 1 atm. in tension. Concrete for arch ring was composed of 1 part Portland cement to $2\frac{1}{2}$ parts sand to 4 parts crushed limestone with $\frac{1}{2}$ part limestone screenings. Settlement of arch during construction (4 months) was 8 cm.

All the above-mentioned hinged and semi-hinged arches, besides others which could not be enumerated here, have given excellent satisfaction and have developed no cracks, even though some were founded on piles and others on clay foundations. Age will undoubtedly be beneficial rather than detrimental, which has never been said for iron or steel bridges.

The oldest concrete bridge seems to have been built in Switzerland near Aarau, in 1840, using Roman cement. This bridge has a span of 7.2 m., and a rise of 3 m. Even this cement, which is not as good as most natural cements, has stood the test of time.

With the adoption of three hinges and the evidence just submitted, it will be possible to construct a masonry arch on almost any moderately good foundation and with reasonable assurance against cracks, both during and after construction, all of which should be regarded as a welcome step in advance. This feature also makes it possible to determine the stresses, for any system of loading, with accuracy and certainty, also to stress the material from one-tenth to one-sixth its ultimate strength as obtained from test samples. All these advantages combined in the three-hinged masonry arch place it on a high plane of engineering perfection. It is hoped that this paper may be the means of introducing this form of arch construction into the United States.

In the following, all dimensions are given in the metric system simply for convenience in computations, but the formulas are equally applicable to any other system of units. The abbreviations used are:

1 meter = 1 m. = 3.2809 ft.
1 square meter = 1 $m.^2$ = 10.7641 sq. ft.
1 cubic meter = 1 $m.^3$ = 35.3156 cu. ft.
1 centimeter = 1 cm. = 0.3937 in.
1 kilogram = 1 kgr. = 2.2046 lbs. avoir.
1 atmosphere = 1 atm. = 0.9482 English atm. = 1 kgr. per $cm.^2$:
= 14.223 lbs. per square inch.

Theoretical Deductions.

I.—General Equations for the Three-Hinged Arch.

(*a*) *Applied Forces and Reactions.*—Given the arch, Fig. 1, with hinged joints at A, B and O, acted upon by the forces P_1 and P_2, assumed to represent the resultants of all vertical loads applied respectively to the left and right of the crown O. Required to find the reactions at the hinges A, B and O.

The thrust produced by P_1 on the segment $O\,B$ must pass through the hinge O, the only point of contact between the segments $A\,O$ and $O\,B$. This thrust must also pass through the hinge B, otherwise the segment $O\,B$ would rotate about B. Also, the intersection c_1 of $O\,A$ with the force P_1 must be a point on the line of the reaction produced

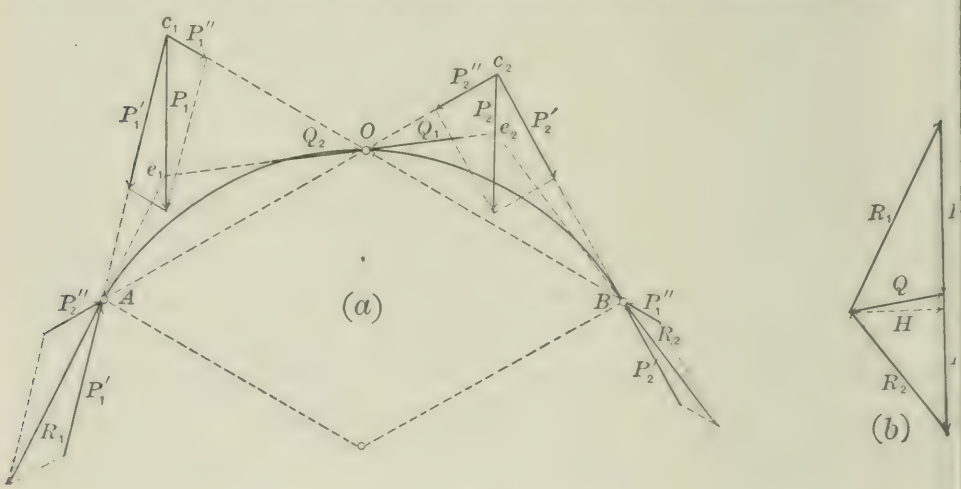

Fig. 1.

by P_1 in A. The reactions in A and B, produced by P_1 are, therefore, P_1' and P_1'', respectively, determined by the resolution of P_1 in the parallelogram indicated.

In like manner the reactions produced by P_2 are found to be P_2'' and P_2', acting in A and B respectively. The resultant reactions in A and B are then R_1 and R_2 respectively, R_1 being the resultant of P_1' and P_2'' and R_2 the resultant of P_1'' and P_2'.

If the forces R_1, R_2, P_1 and P_2 are combined in a force polygon, Fig. 1 *b*, and the equilibrium polygon $A\ e_1\ O\ e_2\ B$ is drawn, it will be seen that the reactions Q are equal and opposite, and that the line of action of $e_1\ O\ e_2$ is a straight line passing through the hinge O. This equilibrium polygon is called the line of thrust for the given forces.

The horizontal components of Q_1, Q_2, R_1 and R_2 are all equal to H, (see Fig. 1 *b*), which is called the horizontal thrust, and is constant for any point of the arch. The vertical components of R_1 and R_2 are the vertical reactions in A and B respectively, and the vertical component V of Q represents the shear at the crown O. Only these horizontal and vertical components of the reactions in A, O and B will be considered in the following, and will be called "the reactions."

The general expressions for the reactions of a three-hinged arch will now be found.

Assume, in Fig. 2, the segment $O\,B$ removed and a force R, resolved into components H (horizontal) and V (vertical), replacing the action of $O\,B$ on $O\,A$. The moment equation about A is

$$P_1 d_1 - Hf + \frac{Vl}{2} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Similarly assume, in Fig. 3, the segment $O\,A$ removed and the

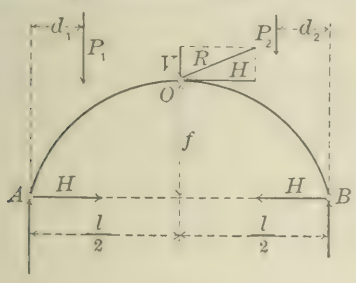

Fig. 2.

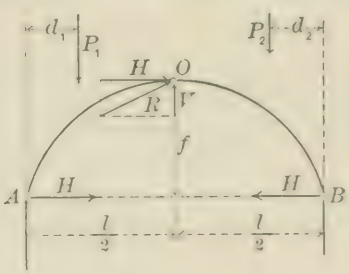

Fig. 3.

equilibrium of segment $O\,B$ preserved by the force R (equal and opposite to R in Fig. 2) resolved into components H and V as before. The moment equation about B is

$$-P_2 d_2 + Hf + \frac{Vl}{2} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

evaluating H and V from (1) and (2)

$$H = \frac{1}{2f}(P_2 d_2 + P_1 d_1) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\text{and } V = \frac{1}{l}(P_2 d_2 - P_1 d_1) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The vertical reactions are obtained as follows from the equations for shear. With reference to Figs. 2 and 3,

$$A = P_1 + V = P_1 + \frac{1}{l}(P_2 d_2 - P_1 d_1) = \frac{P_1 (l - d_1) + P_2 d_2}{l} \ldots (5)$$

$$B = P_2 - V = P_2 - \frac{1}{l}(P_2 d_2 - P_1 d_1) = \frac{P_2 (l - d_2) + P_1 d_1}{l} \ldots (6)$$

From (5) and (6) it will be seen that the reactions A and B are identical with the reactions of a simple beam supported at A and B.

The value of R obtained from either Fig. 2 or Fig. 3 is

$$R = \sqrt{H^2 + V^2} = \sqrt{H^2 + S^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which expression holds good for the resultant thrust at any point of the line of thrust, S being the shear at the point in question.

It follows from (3), that all forces acting on the arch, at points between A and B, affect H positively, while this is not true of the reactions A, B and V.

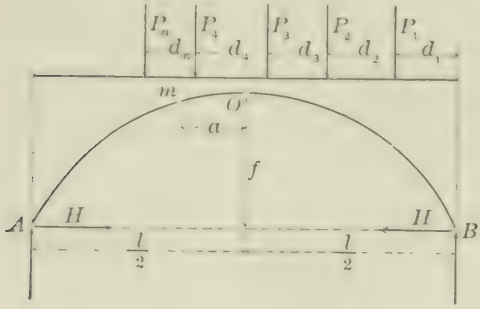

Fig. 4.

(*b.*) *Reactions Resulting from a Train of Concentrated Loads, Coming on the Span from the Right-Hand Abutment.*—The laws indicated by equations (3), (4), (5) and (6) are applied in deducing the expressions for the general case of loading, Fig. 4, as follows:

$$H = \frac{1}{2f}\left[\Sigma_o^B\ P\,d + \Sigma_o^A\ P\,(l-d)\right] \quad \ldots\ldots\ldots\ldots\ldots (8)$$

$$V = \frac{1}{l}\left[\Sigma_o^B\ P\,d - \Sigma_o^A\ P\,(l-d)\right] \quad \ldots\ldots\ldots\ldots\ldots (9)$$

$$A = \frac{1}{l}\ \Sigma_A^B\ P\,d. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$B = \frac{1}{l}\ \Sigma_A^B\ P\,(l-d) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Also, the shear at any point m, distant a from the crown O is found as for a beam of span $A\,B = l$ and is

$$S = A - \Sigma_m^A\ P = \frac{1}{l}\ \Sigma_A^B\ P\,d - \Sigma_m^A\ P \quad \ldots\ldots\ldots\ldots (12)$$

In the above equations the expression $\Sigma_o^B\ P\,d$ is used to indicate the sum of the products $P\,d$ for all loads acting between the points O and B. Other expressions of summations are to be interpreted accordingly.

(*c.*) *Reactions Resulting from a Uniform Live Load p per Unit of Length, Coming on the Span from the Right-Hand Abutment and Extending over the Span to a Distance e to the Left of the Crown O.*—The following equations are obtained analogous to equations (8), (9), (10) and (11):

$$H = \frac{1}{2f}\left[\frac{p\,l^2}{8} + \frac{p\,e\,(l-e)}{2}\right] \quad \ldots\ldots(13)$$

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} - \frac{p\,e\,(l-e)}{2}\right] \quad \ldots\ldots(14)$$

$$A = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 \quad \ldots\ldots(15)$$

$$B = p\left(\frac{l}{2} + e\right) - A = \frac{p}{2\,l}\left(\frac{3\,l^2}{4} + e\,l - e^2\right) \quad \ldots(16)$$

(*d.*) *Symmetrical Loading.*—For equal loads placed symmetrically with respect to the crown, or for symmetrical loading, equations (3), (4), (5) and (6), also equations (8), (9), (10) and (11), give the following special values:

$$H = \frac{\Sigma_o^A\, P\,d}{f}, \qquad V = 0 \quad \text{and} \quad A = B = \Sigma_o^A\, P.$$

For uniformly distributed load over the entire span, $e = \frac{l}{2}$ and equations (13), (14), (15) and (16) give

$$H = \frac{p\,l^2}{8\,f}, \qquad V = 0 \quad \text{and} \quad A = B = \frac{p\,l}{2}.$$

As only symmetrically shaped arches are to be treated in the following, the analysis will be confined to only the half span.

II.—Position of a Moving Load for Maximum Stresses.

For any arch of center line $A\ O\ B$ and hinged at A, O and B, Fig. 5, the reaction at B, resulting from any load P to the left of O, will have the direction $C\ O\ B$ and the reaction at A will pass through A and C. The reaction A will then produce a moment about any point m' of the arch center line, which will be negative or positive accordingly as this point m' is above or below the line $A\ C$, and this moment will become zero for a point m on the line $A\ C$. Hence the vertical through C is the dividing line of loads or load divide for positive and negative influences on the moment about m, which point is the intersection of the line $A\ C$ with the center line $A\ O\ B$, though more properly the line of thrust.

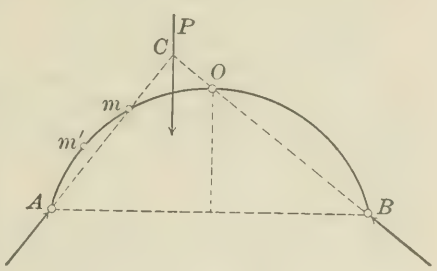

FIG. 5.

Hence, a system of loads covering the span from the right abutment B up to C, will produce maximum compression in the fibers of

the intrados and maximum tension (if any) in the fibers of the extrados for the arch section at m. Also, a system of loads covering the span from the abutment at A up to C will produce maximum compression in the fibers of the extrados and maximum tension (if any) in the fibers of the intrados for this same arch section at m.

Since the resultant thrust in masonry arches is usually confined to the middle third of the arch ring (see Section IV) and accordingly this line of thrust is very nearly normal to the voussoir joints or *radii vectori*, it follows that the shear component of this thrust is necessarily small and any discussion with regard to loading for maximum and minimum shears is considered superfluous, especially as the unit stresses are somewhat liberally chosen, owing to the rather uncertain properties of masonry.

III.—Combined Action of Dead and Live Loads.

(*a.*) *Uniformly Distributed Live Load and Symmetrical Dead Load.*

1. Case of loading for maximum compression in the intrados for any point m of the left half of span: This case of loading will also give the minimum stress in the extrados at m. In accordance with the preceding, the live load must in this case cover the span from the right-hand abutment at D up to the load divide C for the point m, Fig. 6. The following definitions of terms will be adopted and retained throughout this work.

Let m be the point of application of the resultant thrust R of the external forces on any voussoir joint or radial section $m\,n$ under consideration.

Let y and a be the co-ordinates of the point m, referred to the rectangular axes $O\,E$ and $O\,n$ with origin at O.

$\Sigma_o^a\, q$ = resultant dead load of that portion of the span between O and m.

b = distance from O to line of action of $\Sigma_o^a q$.

$\Sigma^{\frac{l}{2}}\, q$ = resultant dead load for the half span $O\,A$.

s = distance from A to the line of action of $\Sigma_o^{\frac{l}{2}}\, q$.

p = live load per unit of length, covering the span from D to C.

e = distance from crown O to the load divide C to the left of O.

H = horizontal thrust from total dead load A to B and live load from D to C.

V = shear at O due to live load D to C. The dead load being symmetrical does not affect V.

S = total shear at m from dead and live loads as assumed for H.

R = resultant thrust at m = resultant of H and S. This resultant passes through m and hence its moment about m must be zero, likewise the sum of the moments of the external forces about m must be zero.

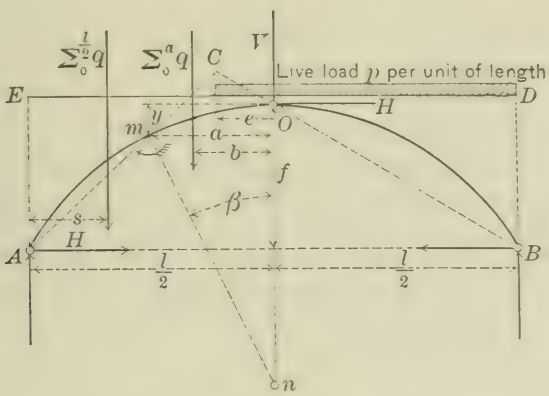

FIG. 6.

Since the dead load is not uniformly distributed in the case of an arch, the quantities, $\Sigma_o^l q$ for successive positions of m are not proportional and must be obtained by summing the q's. The entire arch is divided into vertical segments and the weight q, of each, is determined, as is also the distance, r, from the crown, O, to the center of gravity of each q. The distances, b, are obtained by dividing the sums of the moments, qr, by the sums of the corresponding q's, thus

$$b = \frac{\Sigma_o^a q\, r}{\Sigma_o^a q}.$$

The moment equation of external forces about m is

$$(a - b)\,\Sigma_o^a q + a\,V + e\,p\left(a - \frac{e}{2}\right) - Hy = 0$$

which when solved for y gives

$$y = \frac{(a-b)\,\Sigma_o^a q + a\,V + e\,p\left(a - \frac{e}{2}\right)}{H}$$

wherein, from equations (13) and (14),

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} - p\,e\,\frac{(l-e)}{2}\right] \qquad \text{.....(17)}$$

and

$$H = \frac{s}{f}\,\Sigma_o^l q + \frac{1}{2f}\left[\frac{p\,l^2}{8} + p\,e\,\frac{(l-e)}{2}\right]$$

Also, by analogy, with equations (12) and (15),

$$S = A - \Sigma_a^l q = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \Sigma_o^l q - \Sigma_a^l q = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \Sigma_o^a q \qquad \text{.....(18)}$$

Now, since for any point, m, whose absissa, a, is known or assumed, the ordinate, y, can be found from (17), the locus of m, for all points from A to O, is fully determined and can be drawn. This locus represents the extreme positions of all possible lines of thrust resulting from the combined action of dead load and moving live load.

The loci of maximum and minimum effects give at once the data for obtaining the thickness of arch ring for every assumed point, m (see Section IV).

2. Case of loading for maximum compression in the extrados for any point m of the left half of span. The same loading will also give the minimum stress in the intrados at m. In this case the live load must cover the span from the left hand abutment E up to the load divide C, Fig. 7.

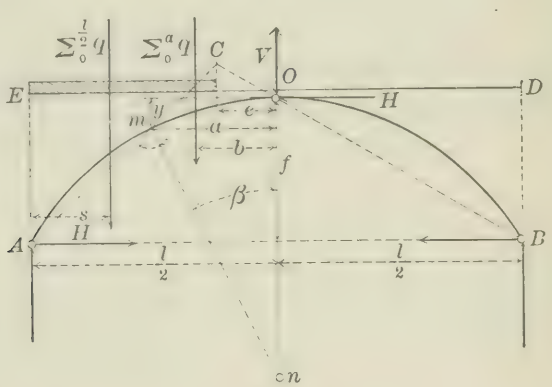

FIG. 7.

Retaining the previous notation, the moment equation for the point m is:

$$(a-b)\,\Sigma_o^a\, q - a\,V + \frac{p}{2}(a-e)^2 - H\,y = 0$$

which gives for y the value

$$y = \frac{(a-b)\,\Sigma_o^a\, q - a\,V + \frac{p}{2}(a-e)^2}{H} \Bigg\} \quad \ldots\ldots\ldots (19)$$

in which

$$V = \frac{p}{2\,l}\left(\frac{l}{2} - e\right)^2 \text{ and } H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{p}{4f}\left(\frac{l}{2} - e\right)^2 \Bigg\}$$

also

$$S = \Sigma_o^{\frac{l}{2}}\, q + \Sigma_o^a\, q + p\,(a-e) - B = \Sigma_o^a\, q + p\,(a-e) - \frac{p}{2\,l}\left(\frac{l}{2} - e\right)^2 \Bigg\} \quad \ldots\ldots (20)$$

3. Case of loading for the half span covered with uniformly distributed live load for any point m of the left half of span. In this instance the locus of y represents a line of thrust for the assumed case of loading. The condition imposed makes $e = 0$, hence for load extending from D to O, (17) and (18) give

$$y = \frac{(a-b)\,\Sigma_o^a\, q + \frac{a\,p\,l}{8}}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

and

$$S = \Sigma_o^a\, q + \frac{p\,l}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

and for load extending from E to O, (19) and (20) give

$$y = \frac{(a-b)\,\Sigma_o^a\, q + \frac{p\,a^2}{2} - \frac{a\,p\,l}{8}}{H} \quad \ldots\ldots\ldots\ldots (23)$$

and

$$S = \Sigma_o^a\, q + a\,p - \frac{p\,l}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

In equations (21) and (23) the value of H remains constant for any position of m and has the value

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l^2}{16\,f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

This condition of loading was formerly applied as a case for maximum stresses at the quarter points; but as is readily seen from the above, this assumption gives values which are much too small.

4. Case of loading for maximum values of H and S, the entire span being symmetrically covered with uniformly distributed live load. For this condition of loading, the shear V at the crown O becomes zero, and the locus of m represents a line of thrust for the imposed loads.

The equation of moments about any point m on the line of thrust is

$$(a-b)\,\Sigma_o^a\, q + \frac{a^2 p}{2} - H\,y = 0\,;$$

which gives

$$y = \frac{(a-b)\,\Sigma_o^a\, q + \frac{a^2 p}{2};}{H}$$

in which

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l^2}{8f} \qquad \text{.....(26)}$$

also

$$S = \Sigma_o^a\, q + a\,p \text{ and } S_{max} = \Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l}{2} = A_{max} \text{.....(27)}$$

(*b.*) *Train of Concentrated Live Loads and Symmetrical Dead Load.*

1. Case of loading for maximum compression in the intrados for any point m of the left half of span. Using the notation given under (*a*) and assuming a system of loads as in Fig. 8, extending over the span from D to the load divide C, the following equations are derived in the manner already indicated for (17) and (18). The same case of loading will also give the minimum stress in the extrados at m.

$$y = \frac{(a-b)\,\Sigma_o^a\, q + a\,V + \Sigma_o^e\, P\left(\frac{l}{2} + a - d\right)}{H}$$

in which

$$V = \frac{1}{l}\left[\,\Sigma_o^{\frac{l}{2}}\, P\,d - \Sigma_o^e\, P\,(l-d)\right] \qquad \text{....(28)}$$

and

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{1}{2f}\left[\,\Sigma_o^{\frac{l}{2}}\, P\,d + \Sigma_o^e\, P\,(l-d)\right]$$

also

$$S = A - \Sigma_a^{\frac{l}{2}}\, q = \frac{1}{l}\,\Sigma_e^{\frac{l}{2}}\, P\,d + \Sigma_o^{\frac{l}{2}}\, q - \Sigma_a^{\frac{l}{2}}\, q = \Sigma_o^a\, q + \frac{1}{l}\,\Sigma_e^{\frac{l}{2}}\, P\,d \text{ (29)}$$

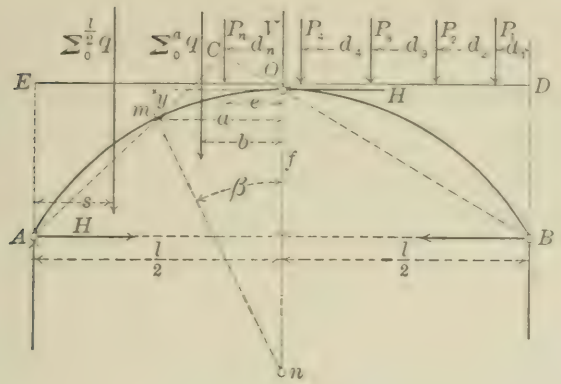

FIG. 8.

It will be seen from the value of V in (28) that a load P falling ex-

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Page 44, last line: The last function of equation (29) should read

$$\frac{1}{l}\,\Sigma_{e}^{\frac{l}{2}}\,P\,d.$$

in which

$$V=\frac{1}{l}\,\Sigma_{e}^{\frac{l}{2}}\,P\,(l-d) \qquad \left.\begin{array}{l}\\ \\ \end{array}\right\}\ \ldots\ldots\ (30)$$

and

$$H=\frac{s}{f}\,\Sigma_{o}^{\frac{l}{2}}\,q+\frac{1}{2f}\,\Sigma_{e}^{\frac{l}{2}}\,P\,(l-d)$$

also

$$\left.\begin{array}{l} S=\Sigma_{o}^{\frac{l}{2}}\,q+\Sigma_{o}^{a}\,q-B+\Sigma_{e}^{a}\,P=\Sigma_{o}^{a}\,q+\Sigma_{e}^{a}\,P \\ \qquad\qquad -\dfrac{1}{l}\,\Sigma_{e}^{\frac{l}{2}}\,P\,(l-d) \end{array}\right\}\ \ldots\ldots\ldots\ldots(31)$$

In order to obtain the greatest load effects in this and the previous cases, the heaviest loads should be placed near O in case 1, and at C in case 2.

(*c.*) *Loading as Under* (*a*) *Combined with a Concentrated Live Load W.*

1. Case of loading for maximum compression in the intrados for any point m of the left half of span, giving also the minimum stress in the extrados at m. Since a single concentrated load exerts its

maximum positive influence on y when applied just to the right of the crown O (as may be seen from equations (28)), the load W will be assumed to act at a unit distance to the right of O for any point m of the left half of span. Hence equations (17) and (18) will apply here when the effect of W is introduced (see Fig. 6).

$$y = \frac{(a-b)\,\Sigma_o^a q + a\,V + e\,p\left(a - \frac{e}{2}\right)}{H}$$

in which

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2} - 1\right) - \frac{e\,p}{2}\,(l-e)\right] \quad \left.\right\} \ldots (32)$$

and

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2f}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2} - 1\right) + \frac{e\,p}{2}\,(l-e)\right]$$

Also

$$S = \Sigma_o^a q + \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \frac{W}{l}\left(\frac{l}{2} - 1\right) \ldots\ldots\ldots (33)$$

2. Case of loading for maximum compression in the extrados for any point m of the left half of span, producing also the minimum stress in the intrados at m. Here W exerts its maximum influence on y when applied vertically over m, and equations (19) and (20) are modified as follows. See, also, Fig. 7.

$$y = \frac{(a-b)\,\Sigma_o^a q - a\,V + \frac{p}{2}\,(a-e)^2}{H}$$

in which

$$V = \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2} - e\right)^2 + W\left(\frac{l}{2} - a\right)\right] \quad \left.\right\} \ldots\ldots (34)$$

and

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2f}\left[\frac{p}{2}\left(\frac{l}{2} - e\right)^2 + W\left(\frac{l}{2} - a\right)\right]$$

also

$$S = \Sigma_o^a q + p\,(a-e) - \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2} - e\right)^2 + W\left(\frac{l}{2} - a\right)\right] \ldots (35)$$

IV.—Conditions of Stress on a Radial Section of an Arch.

(*a*) *The Resultant Normal Thrust on the Section.*—The resultant thrust R at any point m of a linear arch $A\,O$, Fig. 9, is obtained from equation (7), as $R = \sqrt{H^2 + S^2}$, in which H is the horizontal thrust and S the vertical shear for this point. But as R is not usually normal to the radial section $m\,n$, it will have components perpendicular to and parallel with this section.

The resultant R is resolved into N (the normal thrust) and T (the tangential thrust) respectively perpendicular to and parallel with the radial section $m\,n$, which section is represented by the radius vector of the curve $A\,O$ at the point m and makes the angle β with the vertical.

The values of N and T in terms of H and S are now found without involving R in the result.

The forces S, H, R, N and T are shown in their proper relationship in Fig. 9, from which the following equations are obtained:

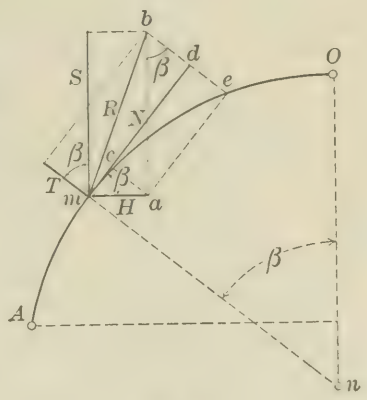

FIG. 9.

The angle $a\,m\,d$ = the angle $a\,b\,d$ = the angle $O\,n\,m = \beta$, also

$$a\,b = S, \text{ and } b\,d = T$$

also

$$a\,e = c\,d = a\,b \sin \beta = S \sin \beta$$

and

$$m\,c = m\,a \cos \beta = H \cos \beta$$

hence

$$N = m\,d = m\,c + c\,d = H \cos \beta + S \sin \beta \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

and

$$T = b\,d = b\,e - e\,d = S \cos \beta - H \sin \beta \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

However, the tangential force T rarely becomes sufficiently large to require any consideration, especially when an arch is so designed that no tensile stresses will ever occur, thereby confining the thrust R to the middle third of the arch-ring and reducing the angle between R and N to a very small quantity. Also the high factors of safety (6 to 10) used in masonry arches would not warrant the consideration of so small a factor as T.

(*b*) *The Stresses on the Section.—1. Analytical Solution.* In Fig. 10, let $A\,B = D$ represent the thickness of the arch on a radial section $m\,n$; also let g be the middle point of $A\,B$, and the point of application of the resultant thrust N, distant w from g. Required, the manner in which N is distributed over the section and the intensities of the unit stresses k_1 and k_2 on the extreme elements of the arch-ring.

If N is given for an arch of unit breadth then k_0, k_1 and k_2 will be unit stresses.

For N acting in g, it is evident that the stress is uniformly distributed over $A\,B$ and $k_0 = k_1 = k_2 = \dfrac{N}{D}$. For N acting at a distance

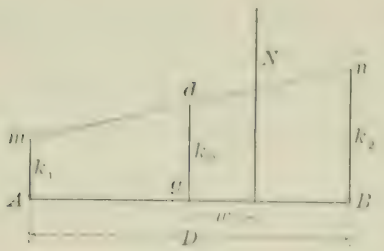

Fig. 10.

w to the right of g, k_1 will be less than k_0, and k_2 will be greater than k_0, each by an amount $f = \frac{M\,y}{I}$ representing the effect of the moment $N\,w$.

Therefore,

$$k_1 = k_0 - \frac{M\,y}{I} \text{ and } k_2 = k_0 + \frac{M\,y}{I} \quad ..(38)$$

in which

$$y = \frac{D}{2},\ I = \frac{b\,h^3}{12} = \frac{D^3}{12} \text{ for } b = 1 \text{ and } M = N\,w = w\,Dk_0$$

By substituting these values in 38 and reducing

$$k_1 = k_0\left(1 - \frac{6\,w}{D}\right) \text{ and } k_2 = k_0\left(1 + \frac{6\,w}{D}\right) \ldots\ldots\ldots (39)$$

The line $m\ d\ n$ represents the manner of distribution of stress produced by the resultant N on the section $A\ B$.

2. *Graphical Solution.* Equations (39) may be written thus, $k_1 = \frac{6\,k_0}{D}\left(\frac{D}{6} - w\right)$ and $k_2 = \frac{6\,k_0}{D}\left(\frac{D}{6} + w\right)$, in which form they may be represented graphically.

In Fig. 11, draw to scale of forces $k_0 = \frac{N}{D}$ at g, perpendicular to $A\ B$, and lay off distances $e\ g$ and $b\ g = \frac{D}{6}$ to the right and left from g. Draw $e\ d$ and prolong same to intersect N in h, also draw $d\ b$ intersecting N in i. Then will $i\ t = A\ m = k_1$, and $t\ h = B\ n = k_2$, and $m\ d\ n$ will represent the law of variation of stress over the section $A\ B$.

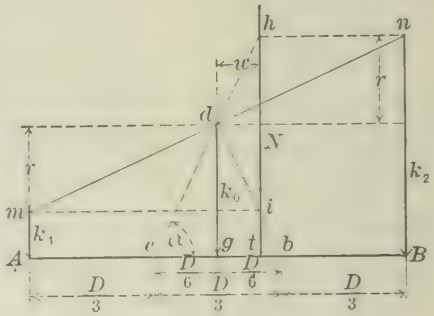

Fig. 11.

$$\text{For } k_1 = k_0 - r = t\,b\tan\alpha = \frac{6\,k_0}{D}\,t\,b = \frac{6\,k_0}{D}\left(\frac{D}{6} - w\right)$$

$$\text{and } k_2 = k_0 + r = e\,t\tan\alpha = \frac{6\,k_0}{D}\,e\,t = \frac{6\,k_0}{D}\left(\frac{D}{6} + w\right).$$

In equations (39) when $w = 0$, $k_1 = k_2 = k_0$, and when $w = \frac{D}{6}$, $k_1 = 0$ and $k_2 = 2\,k_0$. When $w > \frac{D}{6}$, k_1 becomes negative or tensile. Hence

to avoid tensile stresses on any section $A\,B$, the resultant normal thrust N must have its point of application t within the middle third $e\,b$ of said section.

(c) *Thickness of Arch Ring.*—Given the direction, amount and point of application of the normal thrusts N_e and N_i, obtained from the loading for maximum compression in the extrados and intrados respectively, for a radial section $A\,B$, Fig. 12, to find the thickness D of the arch ring which must be provided so that a certain assigned unit stress k on the extreme elements of the ring shall never be exceeded.

While N_e and N_i can never occur simultaneously, the center line of the arch must be so placed, with respect to these thrusts, and a minimum value of D, that the above conditions may be complied with.

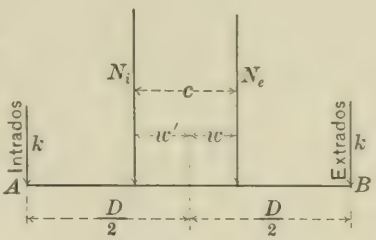

FIG. 12.

The dimension c in Fig. 12 is obtained from the difference between maximum and minimum y, which vertical difference is projected on the radial section $A\,B$ by multiplying with $\cos\beta$; hence $c = \triangle y \cos\beta$ is a known quantity, and w and w' are to be found, likewise D.

The values k_o in equations (39) for the two thrusts are $k_o = \frac{N_e}{D}$ and $k_o' = \frac{N_i}{D}$, and from the figure $w' = c - w$. These values substituted in the second equation (39) give the two following values of $k_2 = k$, the given allowable compressive stress:

$$k = \frac{N_e}{D}\left(1 + \frac{6\,w}{D}\right) \text{ and } k = \frac{N_i}{D}\left(1 + \frac{6\,(c-w)}{D}\right)$$

These equations when solved for D and w give the dimensions sought.

$$D = \frac{N_i}{k\left(1 + \frac{N_i}{N_e}\right)} + \sqrt{\frac{6c}{k}\left[\frac{N_i}{1 + \frac{N_i}{N_e}}\right] + \left[\frac{N_i}{k\left(1 + \frac{N_i}{N_e}\right)}\right]^2} \quad \ldots \quad (40)$$

$$w = \frac{k\,D^2}{6\,N_e} - \frac{D}{6} \quad \ldots \quad (41)$$

The ordinate of the arch centerline for the section $A\,B$ is also found from w, and the ordinate to the point of application of N_e which

may be called y_{min}. Calling the ordinate of the center line y_c, and the inclination of $A\ B = \beta$ from the vertical,

$$y_c = \frac{w}{\cos\ \beta} + y_{min} \ldots\ldots\ldots\ldots\ldots\ (42)$$

The values of k_1 for the above thrusts and dimensions may now be found from the first of equations (39).

(*d*) *Tensile Stresses.*—It may occur that after having found D from (40) and k_1 from (39), the latter value may be negative or tensile. Should this tensile stress be in excess of the allowable stress then the thickness D must be further increased, or if the material of the arch be concrete the excessive tension may be taken up by the insertion of wire netting or iron rods. The method of computing the area of metal required is given in the following:

In Fig. 13, let $A\ B$ be the radial arch section k_1 a tensile stress on the extreme element at A and k_2, the corresponding compressive stress at B.

$a\ b$ is a steel wire placed at a distance v from the axis of the arch. Let a be the area of the steel required for an arch ring of unit breadth and $f =$ the allowable unit stress for steel.

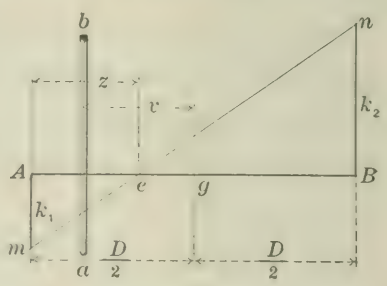

FIG. 13.

The total tensile stress on the arch ring (for unit breadth) will be represented by the area $A\ m\ e = \frac{k_1\ z}{2} = u'$.

From the similar triangles $A\ m\ e$ and $B\ e\ n$

$$\frac{z}{D - z} = \frac{k_1}{k_2}, \text{ or } z = \frac{k_1\ D}{k_1 + k_2}; \text{ making } u' = \frac{k_1^2\ D}{2\ (k_1 + k_2)} \ldots\ldots\ (43)$$

The point of application of u' is the center of gravity of the triangle $A\ m\ e$ which is distant from g by an amount $\frac{D}{2} - \frac{z}{3}$. Hence the moment of u' about g is $M = u'\left(\frac{D}{2} - \frac{z}{3}\right)$ which, when taken up by the steel rod, produces therein the force

$$u = \frac{u'\left(\frac{D}{2} - \frac{z}{3}\right)}{v} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (44$$

But $a = \frac{a'}{f}$; hence from (43) and (44) the value of a is found as

$$a = \frac{k_1^2 D\left(\frac{D}{2} - \frac{k_1 D}{3(k_1 + k_2)}\right)}{2 f r (k_1 + k_2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4$$

In equation (45) k_1 is tensile stress and k_2 is compressive stress, and both enter into the equation without regard to sign of stress.

V.—Deformations of the Arch Ring.

(a) General Considerations; Changes in Length of the Arch Ring.—An arch ring, when under the influence of stress and changes in temperature, is subject to elastic deformations. The compressibility of masonry by stress and the shrinkage caused by the setting process of mortar and concrete, bring about a permanent shortening in the arch ring during construction and test loading.

The resultant effect of these several shortenings and temperature changes produces a deformation of the arch ring which must be provided for in the construction, to prevent a deflection below the normal, which, if it occurred, would materially increase the horizontal thrust on the abutments. This superelevation, which must be given to the arch ring in order that it may attain its proper rise when completed, is called camber.

Besides the above deformation, which is partly elastic and partly permanent, provision must also be made for the settlement in the falsework caused by the weight of the arch ring up to the time of closing, when the latter becomes self-sustaining. This part of the problem is here omitted as depending entirely on the nature of the falsework and local building conditions.

The change in length of the arch ring resulting from stress, temperature, compressibility of material and shrinkage of masonry will now be found.

The normal arch thrust N may be found, for any combination of dead and live loads, from the preceding equations, and this thrust increases from the crown toward the haunches. The cross-section of the arch ring is also a variable quantity. Hence it is necessary to divide the arch ring into sections over which the cross-section and normal thrust may be considered constant. The sum of the increments of change of these sections gives the change in length of the arch ring.

Let L = length of a section of arch, over which the area and thrust are assumed constant.

$\triangle L$ = increase in the length L for any assigned reasons. Decrease is negative.

$\delta = \Sigma_0^{\frac{l}{2}} \triangle L$ = sum of the changes $\triangle L$ from the crown to the abutment.

N = normal thrust acting through the length L.

F = average cross-section of the section of length L.

E = modulus of elasticity of the material for the working stress $\frac{N}{F}$.

E' = modulus of permanent set of the material for the working stress $\frac{N}{F}$

t = a rise, and $-t$ = a fall, in temperature, in degrees Centigrade.

α = coefficient of expansion for 1° Centigrade.

ε = coefficient of shrinkage from setting of mortar or concrete in air.

then
$$\triangle L = -\frac{N L}{E F} - \frac{N L}{E' F} - \varepsilon L + \alpha t L$$

$$= -\frac{N L}{F}\left(\frac{1}{E} + \frac{1}{E'}\right) - L\left(\varepsilon - \alpha t\right) \ldots\ldots\ldots\ldots\ldots (46)$$

and
$$\delta = -\left(\frac{1}{E} + \frac{1}{E'}\right) \Sigma_0^{\frac{l}{2}} \frac{N L}{F} - \left(\varepsilon - \alpha t\right) \Sigma_0^{\frac{l}{2}} L \ldots\ldots (47)$$

The elastic change in length of an arch ring in a completed structure no longer subject to increased permanent set is

$$\delta' = -\frac{1}{E} \Sigma_0^{\frac{l}{2}} \frac{N L}{F} + \alpha t \Sigma_0^{\frac{l}{2}} L \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

(*b*) *Deformation; Analytical Solution.*—To find the deflection at the crown of an arch ring, which latter has shortened an amount δ, as found from (47) or (48).

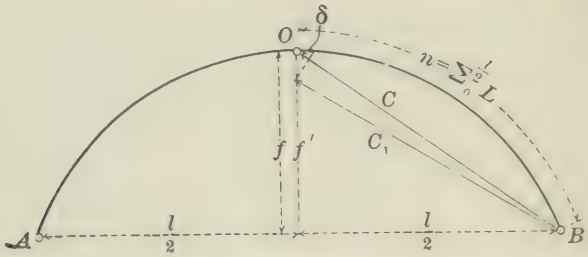

FIG. 14.

Let C, Fig. 14, be the chord from crown to abutment, correspond-

ing to the half span $\frac{l}{2}$ and rise f; C_1 = the length of this chord after the half arch ring has shortened an amount δ and the rise has diminished to f^1, the span remaining unchanged; also, $n = \Sigma \int_0^{\frac{l}{2}} L$ = the length of the center line of the arch ring between the crown and the abutment.

A shortening δ in the length n will produce a shortening $\frac{C}{n}\ \delta$ in the chord C, making $C_1 = C - \frac{C}{n}\ \delta$.

From the figure $C = \sqrt{f^2 + \left(\frac{l}{2}\right)^2}$, which, substituted in the previous equation, gives $C_1 = \left(1 - \frac{\delta}{n}\right)\sqrt{f^2 + \left(\frac{l}{2}\right)^2}$ ..... (49)

and $$\triangle f = f - f^1 = f - \sqrt{C_1^2 - \left(\frac{l}{2}\right)^2} \text{ ..... (50)}$$

The value $\triangle f$, which represents the deflection at the crown, may then be found from equations (49) and (50).

While the value $\frac{C}{n}\ \delta$ is not exactly correct, yet the approximation is so close that the resulting equations give values entirely within the knowable accuracy even for long spans.

As will be noticed, this solution applies to arches of any shape not necessarily circular, but is most accurate for circular arches.

(c) *Deformation; Graphical Solution.*—The general case for any condition of unsymmetrical loading and displacements of abutments can best be solved by the graphical method.* Only the application of the method is here given, without repetition of its derivation, for which see above-named article.

In Fig. 15*a*, let *A O B* represent the line of thrust of a three-hinged arch for any particular case of loading; L_1, L_2, L_3, etc., the sections into which the arch is divided; and $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., the contractions in the lengths L_1, L_2, L_3, etc., respectively, caused from any combination of conditions, as stress, shrinkage of masonry, temperature, etc.

* See article "Distortion of a Framed Structure," by David Molitor, in *Journal of the Association of Engineering Societies*, Vol. xiii, p. 310.

In Fig. 15 *b*, draw in succession, from any point *A'*, the contractions $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., respectively, parallel to the elements L_1, L_2, L_3, etc., of Fig. 15 *a*, and in the direction in which these contractions act relatively to the fixed point *A*. In the example, the $\triangle$'s being negative, the elements of the arch, Fig. 15 *a*, all move toward *A*, and hence the quantities $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., are applied downward from *A'*. The broken line *A' O'* then represents the motion of the point *O* relatively to the point *A*, assuming that the arch elements move parallel to themselves. In like manner, the broken line *O' B'*, Fig. 15 *c*, represents the motion of the point *O* relatively to the point *B*.

However, the elements of the arch do not move parallel to themselves, but the half arch *A O* revolves about *A* and the half arch *O B* revolves about *B* until the point *O*, common to both halves, attains its new position. This revolution is performed by a second operation as follows:

In Fig. 15 *b*, draw *A' O''* perpendicular to *A O* and through *A'*; also through *O'* draw *O' r* perpendicular to *A' O''*, and in Fig. 15*c*, through *B'* draw *B' O''* perpendicular to *B O*, and through *O'* draw *O' s* perpendicular to *B' O''*. Now transfer *O' s* from Fig. 15 *c* to Fig. 15 *b*, parallel and equal to itself, and in Fig. 15 *b* draw *s O''* perpendicular to *O' s*, and *O'' O'* will represent the direction and amount of the displacement of the point *O*. Also transfer *O' r* from Fig. 15 *b* to Fig. 15*c*, and in Fig. 15 *c* draw *r O''* perpendicular to *O' r*, and *O'' O'* will check in direction and amount the value *O'' O'* found in Fig. 15 *b*.

To find the displacements of the individual points 1, 2, 3, etc., draw a broken line *A'*, 1'', 2'', 3'', 4'', *O''* with its elements proportional and respectively perpendicular to the elements *A*, 1, 2, 3, 4, *O*. The lines 1'' 1', 2'' 2', etc., will represent the true displacements of the the points 1, 2, etc., respectively. The same construction applied to Fig. 15 *c* gives the displacements of the points of the half arch *O B*.

When the abutments are displaced by amounts $-\triangle A$ and $-\triangle B$. respectively, these displacements are embodied in the diagrams of Figs. 15, *b* and *c*, as indicated by the dotted construction, and the displacements are then measured from the dotted broken lines O_1'', A_1', and O_1'', B_1', respectively, to the broken lines *O' A'* and *O' B'*.

For symmetrical loading, where the displacements in the half arch *A O* are exactly equal to those in the half arch *B O*, the line of thrust is symmetrical, and the deflection at the crown *O* must be vertical.

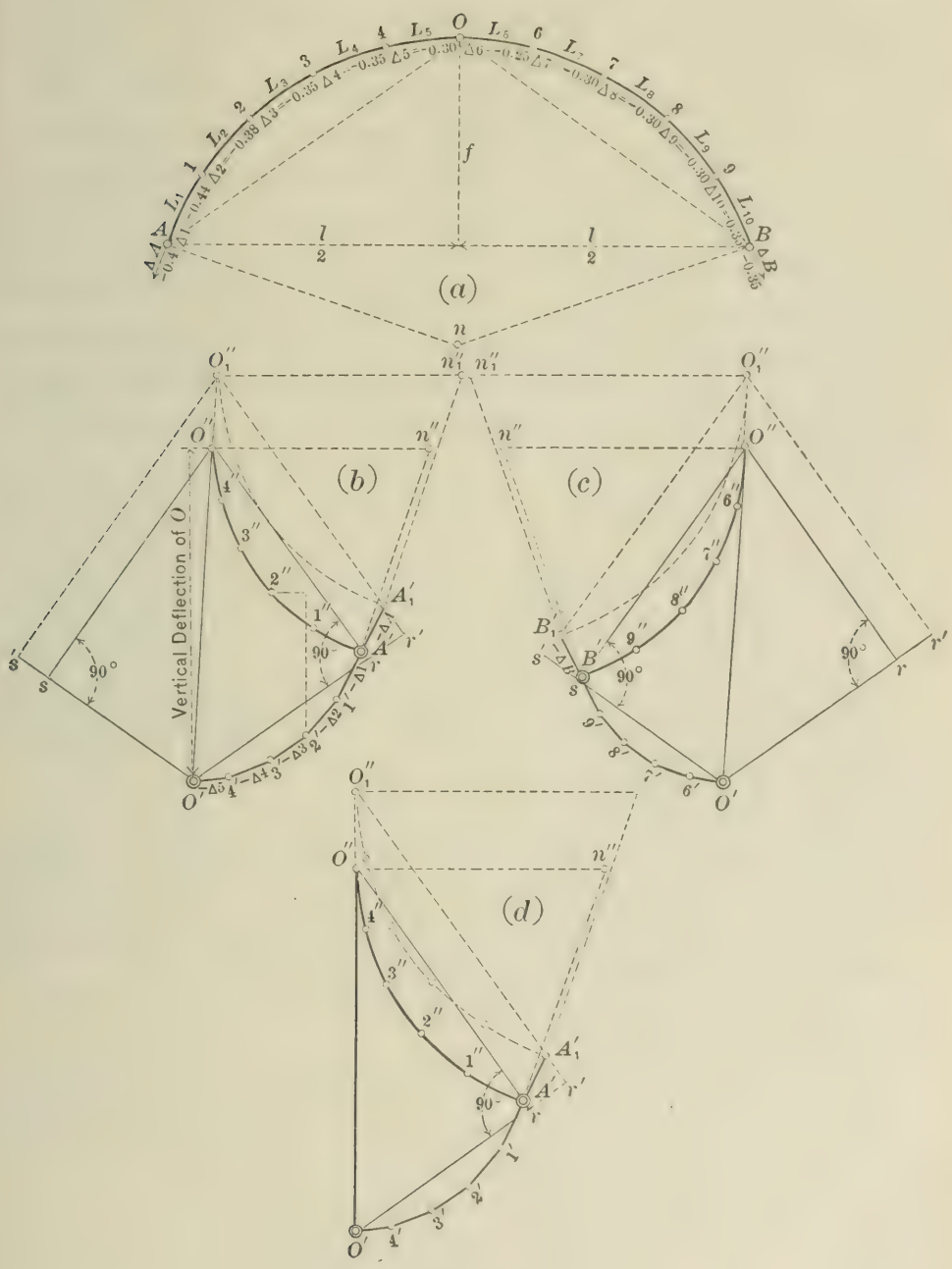

FIG. 15.

This extra condition makes it possible to dispense with one of the diagrams, and the solution becomes as shown in Fig. 15 *d*. The point O'' is then the intersection of $A'\ O''$ perpendicular to $A\ O$, and the vertical $O'\ O''$ through O'. This vertical becomes the deflection at the crown.

It should be noticed in the solution Figs. 15, that Fig. 15 *a* is drawn to a small scale, while the displacement diagrams *b*, *c* and *d* are drawn to natural scale. For small contractions these diagrams may be drawn to a scale of 10 to 100 times the natural.

(*d*) *Values of* ε, α, E *and* E' *in Equations* (*46*) *to* (*48*). While it is always best to make accurate determinations of the above values for the material to be used in any particular structure, especially when close agreement is desired, yet a brief summary of the meagre data on this subject may be useful as furnishing values for preliminary computations.*

Values of ε for German Portland cement mortars, sixteen weeks old.†

Mortar mixed	1	part cement to	0	part sand	$\varepsilon = 0.0012$ to 0.0034
"	1	"	3	"	$\varepsilon = 0.0008$ " 0.0015
"	1	"	5	"	$\varepsilon = 0.0008$ " 0.0014

Values of α for one degree of temperature (Centigrade).

Cement mortar (Bruniceau) ..... $\alpha = 0.00001$ to 0.000014

Portland cement concrete $1:2\frac{1}{2}:5$ (Bauschinger) ..... $\alpha = 0.0000088$

Stone and brick (Bruniceau) ..... $\alpha = 0.0000053$ to 0.0000083

Regarding the values of E and E' it will be necessary to recite briefly the results of the interesting set of experiments made in 1894 by Professor C. Bach, of Stuttgart, which are of vital importance to the subject here treated.

These experiments showed that any concrete when subjected to stress would undergo a permanent set and an elastic deformation, the magnitudes of which become constant after several repetitions of the same stress. The number of repetitions necessary to produce constant deformations appeared to be a function of the breaking strength of the concrete, and of the intensity of the applied stress. The greater the ultimate strength, the less repetitions were required; and the greater the applied stress, the greater the number of repetitions.

* See article by David Molitor on "Properties of Concrete under Compressive Stress," *Jour. Assoc. Eng. Soc.*, May, 1898.

† See report of Committee on Compressive Strength of Cements, etc. *Transactions* Am. Soc. C. E., Vol. xv, p. 717.

However, for each kind of concrete a maximum stress was reached (about 0.7 of breaking stress) for which seven to eight repetitions would still continue to increase the deformations. Presumably more repetitions would have restored constancy, but this point may be regarded as a natural limit of allowable stress, though there does not appear to be a definite limit of elasticity.

The tests were made on cylindrical samples of concrete, 1 m. long and 25 cm. in diameter, mixed 1 part Portland cement to $2\frac{1}{2}$ parts sand to 5 parts broken limestone, and 1 part cement to 3 sand to 6 stone, age two to three months. The breaking strength was in every case determined from the cylindrical samples. The strength of the same concrete developed by cubic samples would have been about one and one-half times the strength obtained from the long cylinders.

The following tables give values of E and E' for various values of ultimate strength and applied loads. These values are given in metric atmospheres.

VALUES OF E IN THOUSANDS (Three ciphers should be added to tabulated values).

ULTIMATE STRENGTH IN ATMOSPHERES.		APPLIED LOADS IN ATMOSPHERES, OR KGR. PER SQ. CM.							
Found from cylinders.	Estimated for cubes.	8	10	15	20	25	30	35	40
60	90	228	223	214	204	193	181	171	163
80	120	262	258	246	234	222	210	200	191
100	150	296	290	278	266	255	244	235	224
120	180	320	314	298	286	276	267	257	247
140	210	340	333	317	303	291	280	270	260

VALUES OF E' IN THOUSANDS (Three ciphers should be added to tabulated values).

ULTIMATE STRENGTH IN ATMOSPHERES.		APPLIED LOADS IN ATMOSPHERES, OR KGR. PER SQ. CM.							
Found from cylinders.	Estimated for cubes.	8	10	15	20	25	30	35	40
60	90	1 230	1 180	1 030	910	810	720	630	545
80	120	1 800	1 710	1 500	1 320	1 180	1 020	870	755
100	150	2 500	2 400	2 100	1 880	1 680	1 460	1 300	1 160
120	180	3 400	3 210	2 800	2 500	2 220	2 010	1 820	1 650
140	210	4 400	4 190	3 600	3 160	2 780	2 500	2 280	2 100

The important conclusions arrived at by Tourtay,* regarding compressive properties of masonry, are here given.

* *Annales des Ponts et Chaussées*, 1885, ii, p. 15.

1. The ultimate strength of small cubes of cement mortar is considerably less than the compressive strength of blocks of masonry made with the same mortar.

2. The pressure which crushes the masonry is an inverse function of the thickness of the mortar joint.

3. Stone plates laid loosely upon each other have a much smaller compressive strength than solid cubes.

4. The same stone plates, when cemented together with neat cement grout, possess the same compressive strength as do solid stone samples.

Practical Applications.

Solution of a Problem.

(*a*) *Introductory.*—To illustrate more clearly the method of arch construction as proposed in the foregoing, a problem is solved in sufficient detail to bring out the practical applications of the theory.

In designing metal bridges, the dead load is generally known, the more accurate for the forms most in use. This is not the case with masonry arches for reasons of insufficient experimental data. Also, a steel arch may be designed with any rational center line because tensile stresses can be provided for, while in masonry only compressive stresses are safely permissible; therefore, the shape of a masonry arch is of necessity a function of its loading.

However, with the dead load known, the three-hinged masonry arch can be analysed and dimensioned with the same degree of accuracy, consistent with the nature of the material, as is possible in steel. But, as the dead load can be obtained in no other way than by computation from assumed dimensions, the solution must be reached by successive approximations of dimensions until the loading resulting therefrom produces stresses in the arch which are not in excess of the allowable unit stresses. Much depends on the experience of the designer as to the rapidity and directness with which a solution may be obtained.

The method which it is believed will lead to a solution with a minimum of useless computation is illustrated in the following problem.

(*b*) *Statement of the Problem.*—Design a three-hinged concrete arch of 72-m. span between pin centers, rise, 10 m., width of driveway,

8 m., and footwalks 2 m. wide, on each side of the driveway. The profile of the site is given, and the abutments are to be founded on hard clay. The bridge is to carry an electric motor car weighing 20 000 kgr. (44 000 lbs.) on a track of 1.43 m. (4-ft. $8\frac{1}{2}$-in.) gauge, and a uniformly distributed live load of 400 kgr. per m.^{2} (82 lbs. per square foot).

The maximum allowable working stresses are 40 atm.* (568.8 lbs. per square inch) in compression, and 2 atm. (28.4 lbs. per square inch) in tension, for concrete composed of 1 part Portland cement to 2 parts sand to 3 parts crushed limestone. This concrete must attain the following compressive strengths on 20-cm. cubes: 220 atm. in 28 days, 350 atm. in 6 months, and 500 atm. in 2 years. The tensile strength of mortar composed of 1 part cement to 3 parts sand must be at least 20 atm. in 28 days.

(*c*) *Outline of the Method.* (See Plates I and II)—In this as in all bridge designs, the details of the roadway and its supports on the arch ring are first designed. By so doing the dead loads may be computed, involving the weight of the arch ring as the only variable subject to correction.

A diagram of the half span (see dimension diagram, Plate II) is then prepared and the roadway and its supporting columns drawn, to which is fitted an arch ring of seeming good proportions for the given span and rise. Herein, the experience of the designer is practically his only guide, but reference to a completed design or a solution once made will assist wonderfully in making close approximations. It will be seen from the side elevation, Plate I, that the center line (a three-center curve) is almost a complete circular arc, slightly flattened at the crown.

The concentrated dead loads, q, are now computed by using the dimensions scaled from the preliminary diagram. These values, together with values of a, r and e, are tabulated together with computed values of $q\,r$ (see Table No. 1).

Then assuming the maximum case of loading over the entire span, compute S_{max} from equation (27), H_{max} from equation (26) and $N_{max} = H_{max} \cos \beta + S_{max} \sin \beta$, from which may be found the required thickness of arch ring at the crown $D_o = \frac{H_{max}}{k}$ and at the abutments

* Stresses and pressures will be given throughout this problem in metric atmospheres equal to 1 kgr. per cm.^{2} (14.22 lbs. per square inch).

$D_A = D_B = \frac{N_{max}}{k}$. If it appears that the assumed dimensions differ widely from those computed, it is well to make a second approximation before proceeding further with the solution, though no correct idea can yet be formed of the required arch thickness at the quarter points. The arch thicknesses, at the crown and springing, should be chosen about 10% greater than found by computation, since these sections will be adjacent to the hinges and some allowance must be made for unequal distribution of pressure due to friction in the hinged bearings.

When the solution appears satisfactory to this stage, the assumed arch ring should be tested at two or three points between the crown and springing. The maximum thickness of the arch ring will occur at a distance of about 0.3 l from the crown. This and two other sections, distant about 0.15 l and 0.4 l from the crown, should be dimensioned, after which it will usually be possible to proceed to the final design and computation, unless the agreement with the preliminary design was not sufficiently close, in which case the process should be repeated by approximating a new arch ring, using the dimensions last obtained. In this latter case some allowance should be made for the probable effect of the new dead loads. The larger the ratio between dead and live loads, the less will be the divergence $\triangle y$ of maximum and minimum thrusts, though an increase in dead loads will also increase the resultant normal thrust N, on any section.

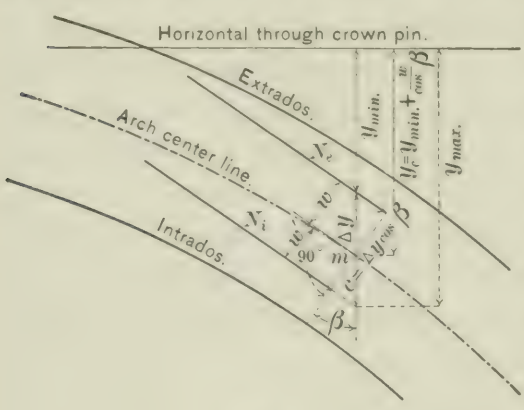

FIG. 16.

The method of dimensioning just referred to consists in computing maximum and minimum y's and the corresponding H's for each of the three points, using in the present problem equations (32) and (34). The differences of the y's give the $\triangle y$'s in Fig. 16. Then, finding S from equations (33) and (35) for each of the three points, compute N_e and N_i from equation (36). From equations (40), (41) and (42) the values D, w and y_c, respectively, may be found. The value of c in equation (40) is equal to $\triangle y \cos \beta$, as may also be seen in Fig. 16.

PLATE I.
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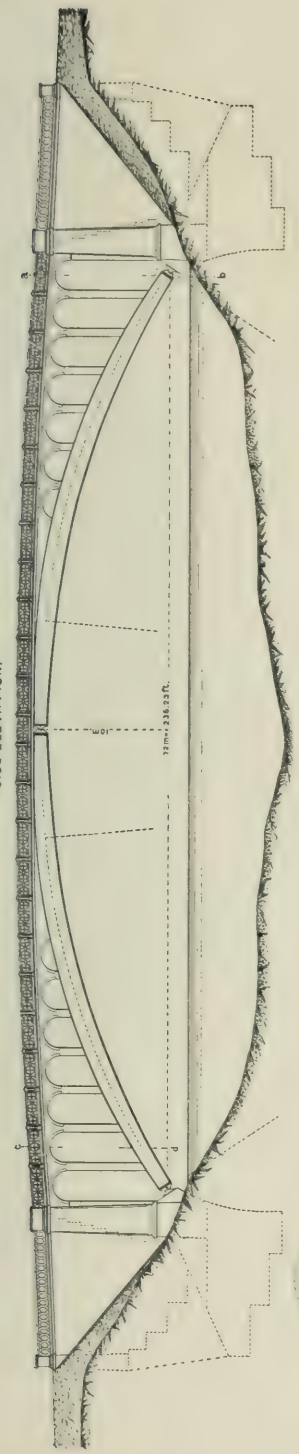
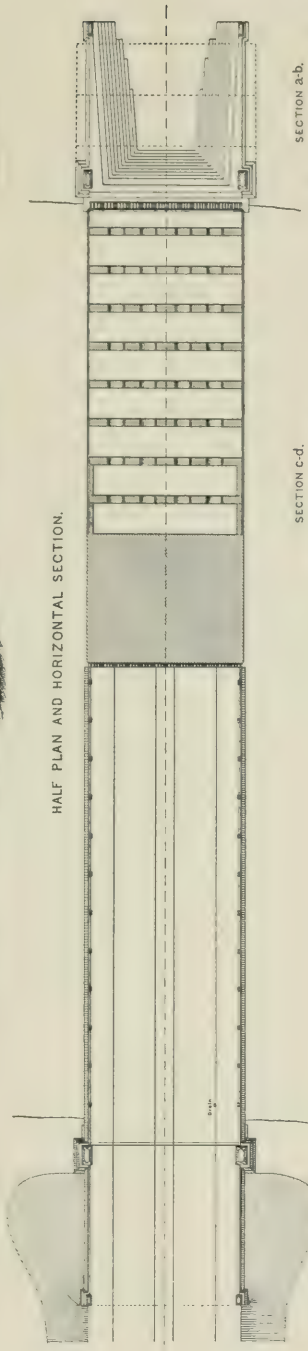
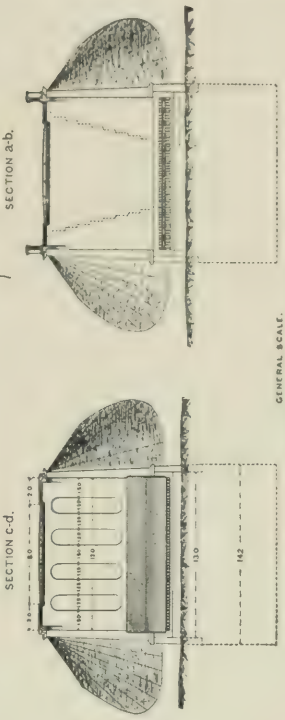
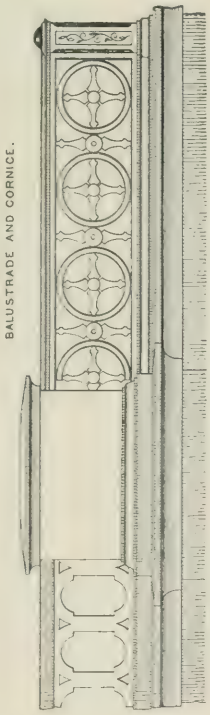
VOL. XL, No. 834.
MOLITOR ON THREE-HINGED MASONRY ARCHES.

DESIGN FOR A THREE-HINGED CONCRETE ARCH.
OF 72 METERS SPAN.
BY DAVID MOLITOR, MEM. AM. SOC. C.E.


SIDE ELEVATION.


HALF PLAN AND HORIZONTAL SECTION.




BALUSTRADE AND CORNICE.

All the factors for five points, including crown and springing, being now fully determined, the intermediate values can easily be interpolated in making the final dimension diagram.

The new diagram is now accurately drawn, and the design carefully computed, in the manner just outlined for the three test points, by application of the method to as many points as may be deemed necessary or desirable to fully determine the dimensions and stresses throughout the arch.

There is still one other point which is interesting to know and is illustrated in the final computation. The center line of the arch ring and the line of thrust for the case of one-half the uniformly distributed live load or $\frac{p}{2}$ acting over the entire span, are so nearly alike that no appreciable error would be introduced by accepting this line of thrust for the center line.

The above outline of the method to be followed is now applied to the problem in hand. In the following computations, the arch ring is assumed 1 m. wide, and the live load thus becomes 400 kgr. per meter, combined with a concentrated live load of $W = 10\,000$ kgr.

(*d.*) *Design of the Roadway.*—From æsthetic considerations and for economic reasons, the roadway is designed to the parabolic curve whose equation is $-y = 0.001\,x^2$, making the same 1.3 m. higher at the crown than at the abutments (see Plate I).

The roadway is made up of concrete floor arches of 2.4 m. span, carried by small concrete piers resting on the arch ring. The cross-section of the driveway is the arc of a circle having a middle ordinate of 0.12 m., and is covered with 6 cm. of asphalt composition. The footwalks are sloped toward the bridge axis with a slope of 1 : 100, and are finished in cement mortar. The car track is placed in the center of the driveway and hard paving brick laid adjacent to the rails.

The horizontal thrust of the floor arches is taken up by steel rods 2.5 cm. in diameter, and spaced 17 cm. apart under the driveway and 75 cm. under the footwalks. Expansion joints are provided in the roadway at the crown and at the abutments, all as shown on Plate II. Drainage of the roadway is provided by cast-iron pipes placed in the first piers adjacent to the abutments, one for each gutter.

In accordance with this design the weight of the floor, complete, to the springing of the arches and between the pier centers, is estimated as follows:

Concrete for arch and portion of pier to springing, 0.57 m.^{3} at 2 300 kgr.	1 311 kgr.
Six steel rods, 2.5 cm. diam., 3 m. long.	71 "
Rails and I-beam	105 "
Six cm. asphalt composition at 2 200 kgr. per m.^{3}	396 "
Total weight to be carried by one pier	1 883 kgr.

The round number 1 900 kgr. was used.

The floor arches are designed to carry 10 000 kgr. evenly distributed over the half span (unsymmetrical load $q + p$, diagram Plate II), and the center line corresponds to the line of thrust following from 10 000 kgr. distributed over the entire span (loads $q + \frac{p}{2}$). The lines of thrust for these loads are drawn for arch horizontal, also for the inclined position between piers 7 and 8. The maximum horizontal thrust is 9 m.^{3} of concrete or 20 700 kgr. for an arch 1 m. wide. The maximum compression at the crown is 17 atm. and at the haunches and quarter points it is 36.8 atm. No tension occurs anywhere.

The longitudinal section investigated is at the curb of the driveway where the minimum thickness of arch (15 cm.) is possible. However, this portion of the roadway, according to the above computation, is amply strong to carry a 20-ton road roller. As a result of the crowning of the roadway the floor arches attain a thickness of 27 cm. in the axis of the bridge.

(*e*) *Design of the Arch Ring.*—A preliminary diagram similar to that shown on Plate II, is now drawn and the dead loads q are computed and tabulated, together with values of a, e, r and β, as far as these may be necessary for the points 3, 6 and 10 in the present problem.

The points 4, 7 and 10 would have been somewhat better, but those chosen are quite suitable for good interpolations of intermediate values.

The preliminary investigation for the crown, the points 3, 6 and 10, and the springing, is given in Table No. 1, the results of which indicate fully to what extent the assumed design was in error.

In the preliminary diagram certain values of D and y_c were assumed. These are tabulated, together with the results from Table No. 1.

PLATE II.
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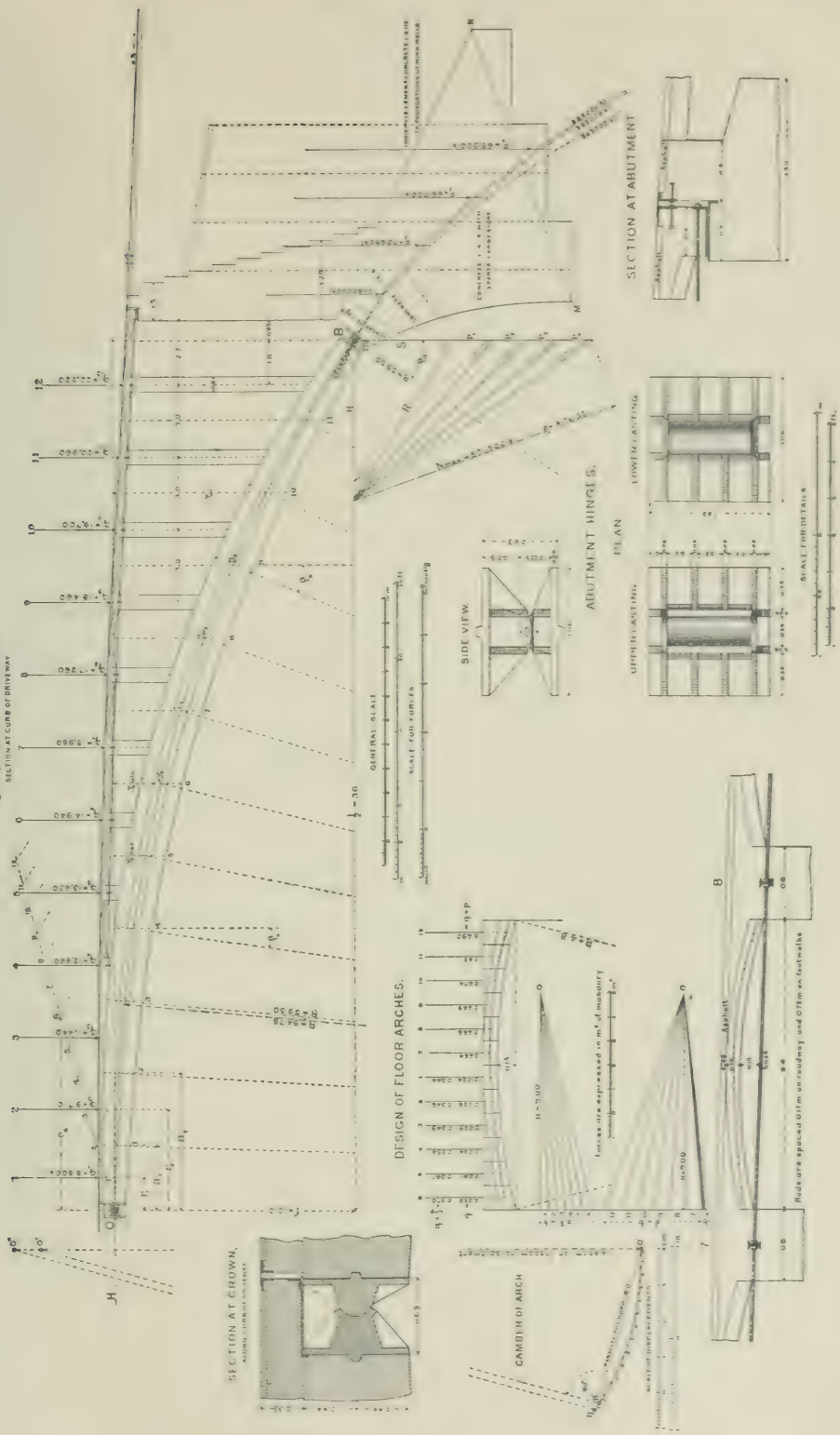
OF 72 METERS SPAN

BY DAVID MOLITOR, MEM. AM. SOC. C.E.



Point.	Preliminary Values.		Computed Values.	
	D	y_c	D	y_c
	m.	m.	m.	m.
0	0.82	0	0.880	0
3	1.40	0.38	1.436	0.423
6	1.63	1.90	1.660	1.964
10	1.40	6.30	1.500	6.394
12	1.00	10.00	1.060	10.000

From this comparison, which shows a remarkably close first approximation of shape and thickness of arch ring, it will be seen that it is now perfectly safe to proceed to the final design and the detailed computation thereof. This could have been done even with a less satisfactory coincidence in the above values of D and y_c.

From the values of D and y_c just found in Table No. 1, the intermediate values must now be interpolated, preparatory to constructing the final dimension diagram on Plate II.

To find the arch center line, plot the five points, whose co-ordinates a and y_c are now known, and connect them by circular arcs using the method indicated in Fig. 17. Connect the plotted points by straight lines forming the polygon $O - 3 - 6 - 10 - 12$. Then inscribe in any circle with convenient center $\times$ on the vertical, through O, a polygon $O' - 3' - 6' - 10' - 12'$, having its sides respectively parallel to those of the foregoing polygon. The radii $\times - 3'$, $\times - 6'$, etc., will be parallel to the required radii $\times_3 - 3$, $\times_6 - 6$, etc., and the latter, when drawn, will intersect in the centers $\times_3$, $\times_6$, $\times_{10}$ and $\times_{12}$ of the arcs sought. Usually a three-center curve will be found to fit an arch of this type, as was done in the present example (see Plate II).

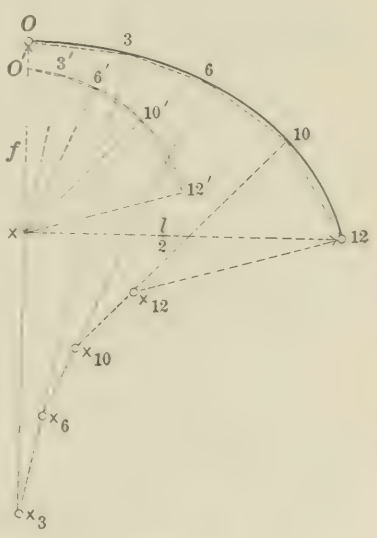


Fig. 17.

To interpolate values of D between those already found, plot the found values as ordinates with the corresponding values of a as in Fig. 18, choosing a vertical scale about twenty times the horizontal. By joining these points with the use of an irregular curved ruler, and noting that maximum D should occur at $0.3\ l$ from the crown, or at

about point 7, the intermediate values of D may then be scaled from the diagram with considerable accuracy.

The diagram, Plate II, can now be drawn, and all necessary dimensions for the final computation are then scaled therefrom. Any slight differences that may be found between these scaled dimensions and the final computed arch dimensions will be too small to warrant a reconsideration, especially as the knowable accuracy with which the dead loads can be determined is far less than any differences still to be expected at this stage of the solution. The final drawings should, of course, be constructed with the use of the dimensions resulting from the final computation.

The final computation is carried out in complete detail in Tables Nos. 2, 3, 4 and 5, and all the steps in the solution of the formulas used are readily traced without any further description. The weights q are based on an assumed weight of the concrete of 2 300 kgr. per cubic meter.

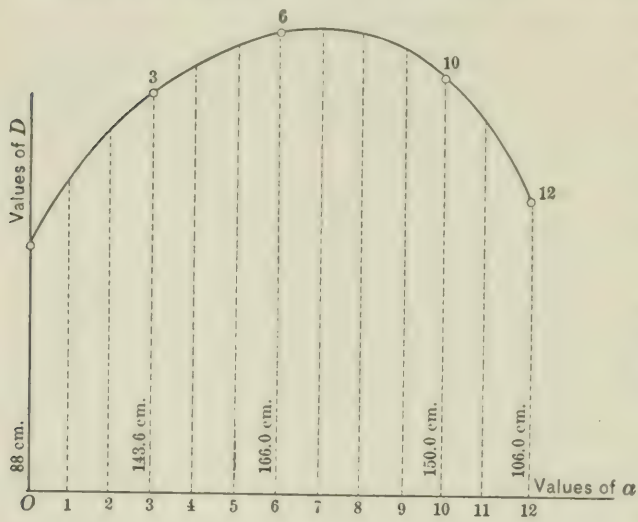


FIG. 18.

Table No. 2 gives the general data relative to dead loads, and the computation of the line of thrust for a live load of 200 kgr. per square meter over the entire span. This case of loading should give a line of thrust corresponding with the center line of the arch ring.

Table No. 3 gives the computation for the values of maximum y, being the result of loading which produces the maximum compression in the intrados, and the minimum stress in the extrados.

Table No. 4 gives the computation for the values of minimum y, being the result of loading which produces the maximum compression in the extrados and the minimum stress in the intrados.

Table No. 5 gives the computation of the thickness of the arch ring at the several points selected, also of the ordinates of the center line, and the unit stresses in the extreme fibers of the arch ring. See Plate II for graphical representation.

TABLE No. 1.—Preliminary Investigation. Live Load of 400 Kgr. Per Meter, and Concentrated Load of 10 000 Kgr.

Point.	q	r	$q\,r$	$\Sigma\, q$	$\Sigma\, q\, r$	$\frac{\Sigma\, qr}{\Sigma\, q} = b$	e	a	β	From Eqs. (32). H	From Eqs. (32). V	From Eqs. (32). $y_{max.}$	S Eq. (33).	N_i Eq. (36).	From Eqs. (34). H	From Eqs. (34). V	From Eqs. (34). $y\,min.$	S Eq. (35).	N_e Eq. (36).	D Eq. (40).	r Eq. (41).	y_c Eq. (42).
	kgr.	m.	kgr. m.	kgr.	kgr. m.	m.	m.	m.		kgr.	kgr.	m.	kgr.	kgr.	kgr.	kgr.	m.	kgr.	kgr.	cm.	cm.	m.
1..	8 000	1.35	10 800	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2..	9 120	4.2	38 304	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
3..	10 840	7.2	78 048	27 960	127 152	4.55	4.2	8.7	5° 40′	312 198	7 670	0.621	37 313	314 320	302 652	6 600	0.207	23 160	303 424	143.6	21.5	0.423
4..	12 400	10.2	126 480	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5..	13 700	13.2	180 840	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6..	14 600	16.2	236 520	68 660	670 992	9.77	8.3	17.7	15° 10′	314 637	6 993	2.266	78 495	324 112	295 713	4 673	1.621	67 747	303 045	166.0	33.0	1.964
7..	15 910	19.2	305 472	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
8..	17 220	22.2	382 284	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
9..	18 120	25.2	456 624	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10..	19 080	28.2	538 056	138 990	2 353 428	16.93	13.0	29.7	26° 30′	317 020	6 330	6.590	150 526	350 550	287 330	2 344	6.150	143 326	320 796	150.0	22.0	6.394
11..	20 320	31.2	633 984	.....	.....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
12..	22 840	34.2	781 128	182 150	3 768 540	20.689	....	36.0	33° 30′	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

$\Sigma_o^{\frac{l}{2}}\, q = 182\,150.$ $\qquad s = 15.311.$

Constants..... $\frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q = 278\,890.$ $\qquad \frac{p\,l^2}{8} = 259\,200.$ $\qquad W\left(\frac{l}{2} - 1\right) = 350\,000.$

Case of maximum loading. Entire span covered with 400 kgr. per meter and 10 000 kgr. at the crown.

Equation (26)..... $H_{max} = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l^2}{8\,f} + \frac{W\,l}{4\,f} = 322\,810$ kgr., and D_o at crown $= \frac{H_{max}}{k} = \frac{3228.1}{40}$ $= 80.7$ cm. $+ 10\% = 88$ cm.

Equation (27)..... $S_{max} = \Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l}{2} + \frac{W}{2} = 201\,550$ kgr.

Equation (36) ..... $N_{max} = H_{max} \cos \beta_{12} + S_{max} \sin \beta_{12} = 394\,760$ kgr., and D_{12} at springing $= \frac{N_{max}}{k} =$ $\frac{3947.6}{40} = 98.7$ cm., or say 106 cm.

TABLE No. 2.—General Data Relative to Dead Loads and Computation of Line of Thrust for Live Load of 200 Kgr. per Square Meter over Bridge.

Point.	q	r	$q\,r$	$\Sigma_o^a q$	$\Sigma q\,r$	$\frac{\Sigma q\,r}{\Sigma q} = b$	a	e	β	$\sin\beta$	$\cos\beta$	$a-b$	$(a-b)\Sigma_o^a q$	$\frac{a^2 p}{2}$	y Eqs. (26).
	kgr.	m.	kgr. m.	kgr.	kgr. m.	m.	m.	m.				m.	kgr. m.	kgr. m.	m.
1.....	8 800	1.35	11 880	8 800	11 880	1.35	2.7	1.4	1° 00′	0.017	0.9998	1.35	11 880	729	0.042
2.....	9 710	4.2	40 782	18 510	52 662	2.86	5.7	2.8	2° 50′	0.050	0.999	2.84	52 568	3 249	0.185
3.....	11 440	7.2	82 368	29 950	135 030	4.51	8.7	4.2	5° 10′	0.090	0.996	4.19	125 490	7 569	0.442
4.....	12 860	10.2	131 172	42 810	266 202	6.22	11.7	5.6	8° 10′	0.142	0.990	5.48	234 599	13 689	0.824
5.....	13 630	13.2	179 916	56 440	446 118	7.90	14.7	6.9	11° 10′	0.194	0.981	6.80	383 792	21 609	1.346
6.....	14 940	16.2	242 028	71 380	688 146	9.65	17.7	8.1	14° 00′	0.242	0.970	8.05	574 609	31 329	2.012
7.....	15 980	19.2	306 816	87 360	994 962	11.39	20.7	9.3	16° 50′	0.290	0.957	9.31	813 322	42 849	2.842
8.....	17 260	22.2	383 172	104 620	1 378 134	13.17	23.7	10.5	20° 10′	0.345	0.939	10.53	1 101 649	56 169	3.844
9.....	18 460	25.2	465 192	123 080	1 843 326	14.98	26.7	11.6	23° 20′	0.396	0.918	11.72	1 442 498	71 289	5.026
10.....	19 700	28.2	555 540	142 780	2 398 866	16.80	29.7	12.7	26° 30′	0.446	0.895	12.90	1 841 862	88 209	6.408
11.....	20 960	31.2	653 952	163 740	3 052 818	18.64	32.7	13.8	29° 50′	0.497	0.867	14.06	2 302 184	106 929	7.998
12.....	22 520	34.2	770 184	186 260	3 823 002	20.525	36.0	14.8	33° 30′	0.552	0.834	15.475	2 882 374	129 600	10.000

$$s = \frac{l}{2} - b_{12} = 15.475$$

Case of maximum loading, for live load $p = 400$ kgr. per square meter over entire span, and $W = 10\,000$ kgr. at crown.

Constants $\quad \frac{s}{f}\ \Sigma_o^{\frac{l}{2}} q = 288\,237 \qquad p\,\frac{l^2}{8} = 259\,200 \qquad W\left(\frac{l}{2} - 1\right) = 350\,000.$

From equation (26). $\quad H_{max} = \frac{s}{f}\ \Sigma_o^{\frac{l}{2}} q + p\frac{l}{8} + \frac{Wl}{4f} = 332\,157$ kgr., from which $D_o = \frac{3321.57}{40} = 83$ cm.

88 cm. was taken.

From equation (27). $\quad S_{max} = \Sigma_o^{\frac{l}{2}} q + p\frac{l}{2} + \frac{W}{2} = 205\,660$ kgr.

From equation (36). $\quad N_{max} = H_{max}\cos\beta_{12} + S_{max}\sin\beta_{12} = 390\,543$ kgr., from which $D_{12} = \frac{3905.4}{40} = 97.6$ cm.

106 cm. was taken.

From equation (37). $\quad T = S\cos\beta - H\sin\beta = -11\,860$ kgr.

TABLE No. 3.—VALUES OF y_{max}, N_i AND T_i.

Point.	$\frac{p\,e}{2\,l}(l-e)$	$\frac{p\,e}{4f}(l-e)$	$\frac{p}{2\,l}\left(\frac{l}{2}+e\right)^2$	$p\,e\left(a-\frac{e}{2}\right)$	$a\,V$	$(a-b)\,\Sigma_o^a\,q.$ from Table No 2.	H	$y_{max.}$	$\Sigma_o^a\,q.$ from Table No. 2.	S	$H\cos\beta$	$S\sin\beta$	N_i	T_i
					kgr. m.	kgr. m.	kgr.	m.	kgr.	kgr.	kgr.	kgr.	kgr.	kgr.
1.....	274.0	986	3 884	2 160	22 105	11 880	319 683	0.113	8 800	17 545	319 621	298	319 919	12 106
2.....	538.3	1 938	4 181	4 816	45 159	52 568	320 635	0.320	18 510	27 552	320 315	1 377	321 692	11 493
3.....	791.0	2 847	4 488	11 088	66 729	125 490	321 544	0.632	29 950	39 299	320 258	3 537	323 785	10 203
4.....	1 032.9	3 718	4 806	19 936	86 909	234 599	322 415	1.059	42 810	52 477	319 191	7 452	326 643	6 169
5.....	1 247.8	4 492	5 111	30 050	106 034	383 792	323 189	1.609	56 440	66 412	317 048	12 884	329 932	2 452
6.....	1 437.8	5 176	5 401	44 226	124 311	574 609	323 873	2.294	71 380	81 642	314 157	19 757	333 914	816
7.....	1 619.8	5 831	5 699	59 706	141 613	813 322	324 528	3.127	87 360	97 920	310 573	28 397	338 970	— 404
8.....	1 793.8	6 458	6 004	73 290	158 013	1 101 649	325 155	4.099	104 620	115 485	305 320	39 842	345 162	— 3 738
9.....	1 946.3	7 006	6 292	96 976	173 942	1 442 498	325 703	5.261	123 080	134 233	298 995	53 156	352 151	— 5 752
10.....	2 092.0	7 531	6 586	118 618	189 159	1 841 862	326 228	6.590	142 780	154 227	291 974	68 785	360 759	— 7 465
11.....	2 231.0	8 032	6 887	142 416	203 721	2 302 184	326 729	8.106	163 740	175 488	283 274	87 217	370 491	—10 236
12.....	2 351.6	8 466	7 166	169 312	219 938	2 882 374	327 163	10.000	186 260	198 287	272 854	109 454	382 308	—15 223

EQUATIONS USED.

Equations.... (32)

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2}-1\right) - \frac{p\,e}{2}(l-e)\right] = 8\,461 - \frac{p\,e}{2\,l}(l-e)$$

$$H = \frac{s}{f}\,\Sigma_{\frac{2}{o}}^{l}\,q + \frac{1}{2f}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2}-1\right) + \frac{p\,e}{2}(l-e)\right] = 318\,697 + \frac{p\,e}{4f}(l-e)$$

$$y_{max} = \frac{(a-b)\,\Sigma_o^a\,q + a\,V + e\,p\left(a-\frac{e}{2}\right)}{H}$$

Equation..... (33) $$S = \frac{p}{2\,l}\left(\frac{l}{2}+e\right)^2 + \frac{W}{l}\left(\frac{l}{2}-1\right) + \Sigma_o^a\,q.$$

Equation..... (36) $$N_i = H\cos\beta + S\sin\beta.$$

Equation..... (37) $$T_i = S\cos\beta - H\sin\beta.$$

DATA.

Values of a, e, $\sin\beta$ and $\cos\beta$ from Table No. 2.

$\frac{l}{2} = 36$ m. and $f = 10.0$ m.

$p = 400$ kgr. per square meter and $W = 10\,000$ kgr.

$\frac{s}{f}\,\Sigma_{\frac{2}{o}}^{l}\,q = 288\,237$ and $\frac{W}{l}\left(\frac{l}{2}-1\right) = 4\,861.$

TABLE No. 4.—VALUES OF y_{min}, N_e AND T_e.

Point.	$\frac{p}{2}\left(\frac{l}{2}-e\right)^2$	$W\left(\frac{l}{2}-a\right)$	V	H	$p\,(a-e)$	$\frac{p}{2}\,(a-e)^2$	$(a-b)\,\Sigma_o^a q$ from Table No. 2.	aV	$y_{min.}$	$\Sigma_o^a q$ from Table No. 2.	S	$H\cos\beta$	$S\sin\beta$	N_e	T_e
			kgr.	kgr.			kgr. m.	kgr. m.	m.	kgr.	kgr.	kgr.	kgr.	kgr.	kgr.
1	239 432	333 000	7 950	316 859	520	338	11 880	21 465	−0.029	8 800	1 370	316 797	23	316 820	−4 016
2	220 448	303 000	7 270	314 409	1 160	1 682	52 568	41 439	+0.047	18 510	12 400	314 095	620	314 715	−3 332
3	202 248	273 000	6 600	311 999	1 800	3 872	125 490	57 420	0.231	29 950	25 150	310 751	2 263	313 014	−3 030
4	184 832	243 000	5 943	309 629	2 440	7 442	234 599	69 533	0.557	42 810	39 307	306 533	5 582	312 115	−5 053
5	169 362	213 000	5 310	307 355	3 120	12 168	383 792	78 057	1.034	56 440	54 250	301 515	10 524	312 039	−6 408
6	155 682	183 000	4 704	305 171	3 840	18 432	574 609	83 261	1.670	71 380	70 516	296 016	17 065	313 081	−5 450
7	142 578	153 000	4 105	303 016	4 560	25 992	813 322	84 973	2.489	87 360	87 815	289 986	25 466	315 452	−3 835
8	130 050	123 000	3 515	300 889	5 280	34 848	1 101 649	83 305	3.500	104 620	106 385	282 535	36 703	319 238	−3 911
9	119 072	93 000	2 945	298 840	6 040	45 602	1 442 498	78 631	4.713	123 080	126 175	274 335	49 965	324 300	−2 512
10	108 578	63 000	2 383	296 816	6 800	57 800	1 841 862	70 775	6.162	142 780	147 197	265 650	65 650	331 300	−1 219
11	98 568	33 000	1 827	294 815	7 560	71 442	2 302 184	59 743	7.848	163 740	169 473	255 605	84 228	339 833	+ 410
12	89 888	000	1 248	292 731	8 480	89 888	2 882 374	44 928	10.000	186 260	193 492	244 138	106 807	350 945	− 215

EQUATIONS USED.

Equations 34..

$$V = \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right]$$

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2f}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right]$$

$$y_{min} = \frac{(a-b)\,\Sigma_o^a q - aV + \frac{p}{2}(a-e)^2}{H}$$

Equation 35... $S = \Sigma_o^a q + p\,(a-e) - V.$

Equation 36... $N_e = H\cos\beta + S\sin\beta.$

Equation 37... $T_e = S\cos\beta - H\sin\beta.$

DATA.

Values of a, e, $\sin\beta$ and $\cos\beta$ are taken from Table No. 2.

$\frac{l}{2} = 36$ m. and $f = 10.0$ m.

$p = 400$ kgr. per square meter and $W = 10\,000$ kgr.

$\frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q = 288.237.$

Point.	Δy.	Cos β. Table No. 2.	c.	N_i Table No. 3.	N_e Table No. 4.	$\frac{N_e}{k\left(1+\frac{N_i}{N_e}\right)}$	$\left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right]^2$	$6c\left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right]$	D.	$\frac{kD}{N_e}-1$	w.	y_{min}. Table No. 4.	y_c	w'	Unit Stresses Caused by N_e: $k_e=\frac{N_e}{D}\left(1+\frac{6w}{D}\right)$	Unit Stresses Caused by N_e: $k_i=\frac{N_e}{D}\left(1-\frac{6w}{D}\right)$	Unit Stresses Caused by N_i: $k_e=\frac{N_i}{D}\left(1-\frac{6w'}{D}\right)$	Unit Stresses Caused by N_i: $k_i=\frac{N_i}{D}\left(1+\frac{6w'}{D}\right)$
	cm.		cm.	kgr.	kgr				cm.		cm.	m.	m.	cm.	atm.	atm.	atm.	atm.
1..	14.2	0.999	14.2	3 200	3 168	39.8	1 584.0	3 390.9	110.4	0.39	7.2	−0.029	0.043	7.0	+39.9	+17.50	+17.97	+40.0
2..	27.3	0.999	27.3	3 217	3 147	39.8	1 584.0	6 519.2	129.8	0.65	14.1	+0.047	0.188	13.2	40.0	+ 8.48	+ 9.67	40.0
3..	40.1	0.996	40.1	3 238	3 130	39.8	1 584.0	9 575.8	145.4	0.86	20.8	0.231	0.440	19.3	40.0	+ 3.00	+ 2.21	40.0
4..	50.2	0.990	50.1	3 266	3 121	39.9	1 592.0	11 993.9	156.4	1.00	26.1	0.557	0.820	24.0	40.0	− 0.06	+ 1.67	40.0
5..	57.5	0.981	56.4	3 299	3 120	40.1	1 608.1	13 569.8	163.3	1.10	29.9	1.034	1.339	26.5	40.1	− 1.89	+ 0.53	39.9
6..	62.4	0.970	60.5	3 339	3 131	40.4	1 632.2	14 665.2	168.0	1.15	32.2	1.670	2.002	28.3	40.0	− 2.80	− 0.20	40.0
7..	63.8	0.957	61.1	3 390	3 154	40.8	1 664.6	14 957.3	169.7	1.15	32.5	2.489	2.828	28.6	40.1	− 2.80	− 0.20	40.0
8..	59.9	0.939	56.2	3 452	3 192	41.4	1 711.0	13 960.1	166.6	1.09	30.3	3.500	3.833	25.9	40.0	− 1.72	+ 1.44	39.9
9..	54.8	0.918	50.3	3 522	3 243	42.2	1 780.8	12 735.9	162.7	1.01	27.3	4.713	5.010	23.0	40.0	− 0.14	+ 3.24	40.0
10..	42.8	0.895	38.3	3 608	3 313	43.2	1 866.2	9 927.4	151.8	0.83	21.0	6.162	6.397	17.3	39.9	+ 3.61	+ 7.60	40.0
11..	25.8	0.867	22.4	3 705	3 398	44.3	1 962.5	5 953.9	133.3	0.57	12.7	7.848	7.994	9.8	40.0	+10.97	+15.57	40.0
12..	00	0.834	00	3 823	3 509	.....	.....	.....	.....	.....	0.0	10.000	10.000	0.0	.....	.....	.....	.....

Equations Used.

$\triangle y = y_{max} - y_{min}$, and $c = \triangle y \cos \beta$.

Equation (40). $$D = \sqrt{6c\left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right] + \left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right]^2} + \frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}$$

Equation (41). $$w = \frac{D}{6}\left(\frac{kD}{N_e} - 1\right)$$

Equation (42). $$y_c = y_{min} + \frac{w}{\cos \beta}$$

Equation (39). $$k_1 = \frac{N}{D}\left(1 - \frac{6w}{D}\right) \text{ and } k_2 = \frac{N}{D}\left(1 + \frac{6w}{D}\right)$$

The values of N_i and N_e in the above table are given for an arch 1 cm. wide, thus giving unit stresses in atmospheres of 1 kgr. per cm.^{2} for k_i and k_e.

w = distance from the point of application of N_e to the arch center line.

$w' = c - w$ = distance from the point of application N_i to the arch center line.

y_c = ordinate of the arch center line.

k = allowable unit stress on the extreme fiber = 40 atm. in compression; 2 atm. in tension (See also Fig. 16).

The pressure on the abutment foundation is given on Plate II.

TABLE No. 6.—Change in Length of the Arch Ring.

Point.	$\Sigma_o^a q$ from Table No. 2.	$a\,p$	S	$H\cos\beta$	$S\sin\beta$	N	F	L	$\frac{N}{F}$	$\frac{NL}{F}$	$\frac{NL}{F}\left(\frac{1}{E}+\frac{1}{E'}\right)$	$L(\varepsilon+\alpha t)$	ΔL.
	kgr.	kgr. m.	kgr.	kgr.	kgr.	kgr.	cm².	cm.	atm.		cm.	cm.	cm.
0.....	000	00	00	301 197	000	301 197	9 360	135	32.2	4 347	—0.0162	—0.1499	—0.1661
1.....	8 800	540	9 340	301 137	159	301 296	11 040	285	27.3	7 780	—0.0290	—0.3163	—0.3453
2.....	18 510	1 140	19 650	300 896	983	301 879	12 980	300	23.3	6 990	—0.0260	—0.3330	—0.3590
3.....	29 950	1 740	31 690	299 993	2 852	302 845	14 580	301	20.8	6 261	—0.0233	—0.3341	—0.3574
4.....	42 810	2 340	45 150	298 185	6 411	304 596	15 640	304	19.5	5 928	—0.0221	—0.3374	—0.3595
5.....	56 440	2 940	59 380	295 474	11 520	306 994	16 330	306	18.8	5 753	—0.0214	—0.3397	—0.3611
6.....	71 380	3 540	74 920	292 161	18 131	310 292	16 800	309	18.5	5 717	—0.0213	—0.3430	—0.3643
7.....	87 360	4 140	91 500	288 246	26 535	314 781	16 970	314	18.5	5 809	—0.0216	—0.3485	—0.3701
8.....	104 620	4 740	109 360	282 824	37 729	320 553	16 660	319	19.2	6 125	—0.0228	—0.3541	—0.3769
9.....	123 080	5 340	128 420	276 499	50 854	327 353	16 270	325	20.1	6 532	—0.0243	—0.3607	—0.3850
10.....	142 780	5 940	148 720	269 571	66 329	335 900	15 180	333	22.1	7 359	—0.0274	—0.3696	—0.3970
11.....	163 740	6 540	170 280	261 138	84 629	345 767	13 330	342	26.9	9 200	—0.0342	—0.3796	—0.4138
12.....	186 260	7 200	193 460	251 198	106 790	357 988	11 280	216	31.7	6 847	—0.0255	—0.2398	—0.2653

Average = 23.0 Total δ = —4.5208

Equations Used.

Equation..(26). $H = \frac{s}{f}\,\Sigma_o^l\, q + \frac{pl^2}{8f} = 301\,197$ kgr.

Equation..(27). $S = \Sigma_o^a\, q + a\,p.$

Equation (36). $N = H\cos\beta + S\sin\beta.$

Equation..(46). $\triangle L = -\frac{NL}{F}\left(\frac{1}{E}+\frac{1}{E'}\right) - L\left(\varepsilon + \alpha t\right)$

A comparison of the last assumed data, and the final dimensions obtained in Table No. 5 is given in the following table, from which it is clearly seen that the last assumed arch ring gave dead loads and dimensions which could scarcely be improved by further computations:

Point.	Assumed Dimensions.		Computed Dimensions.		For ½ total live load as found in Table No. 2.
	$D.$	y_c	$D.$	y_c	$y.$
	cm.	m.	cm.	m.	m.
0	88	0.00	88	0.000	0.000
1	112	0.05	110.4	0.043	0.042
2	131	0.18	129.8	0.188	0.185
3	145	0.45	145.8	0.440	0.442
4	154	0.80	156.4	0.820	0.824
5	162	1.29	163.3	1.339	1.346
6	166	1.96	168.0	2.002	2.012
7	167	2.80	169.7	2.828	2.842
8	166	3.79	166.6	3.833	3.844
9	160	4.98	162.7	5.010	5.026
10	150	6.38	151.8	6.397	6.408
11	134	7.96	133.3	7.994	7.998
12	106	10.00	106.0	10.000	10.000

The above agreement between y_c and y illustrates very strikingly the statement previously made, viz.: that the line of thrust, for a case of entire span covered with one-half the uniform live load, represents practically the center line of the arch.

It will be noticed that the tensile stresses on the intrados at points 6 and 7 (see Table No. 5) are slightly in excess of the allowable, being 2.8 atm., as against 2 atm. allowed. This would probably be safe, but having set the limit at 2 atm., the excess of 0.8 atm. must be provided for, either by giving the section larger dimensions at these points, or by introducing a few iron rods to take up the excessive tension. The latter method will be adopted merely to illustrate the application of the formula.

The sectional area of iron required for an arch of 1 cm. width is by

$$\text{equation (45)},\ a = \frac{k_1^2\, D \left(\frac{D}{2} - \frac{k_1 D}{3\,(k_1 + k_2)}\right)}{2 f v\,(k_1 + k_2)}$$

in which $k_1 = 0.8$ atm., $k_2 = 40$ atm., $D = 169.7$ cm., $f = 700$ atm. (10 000 lbs. per square inch), and $v = 75$ cm., leaving about 10 cm. of concrete outside the iron.

By substituting these values in (45), and solving, it is found that $a = 0.002$ cm.^{2}, or for an arch 1 m. wide 0.2 cm.^{2} would be required,

or 3 rods of 3 mm. diameter for each meter of arch, or 7 rods of 2 mm. diameter, whichever may be preferable.

(*f*) *Design of the Hinged Bearings.*—The radius of curvature of the rolling surface according to Winkler, Heinzerling or Melan is given by $r = \frac{2\ N}{\pi\ k\ l}$ in which N = the maximum normal pressure on the bearing = 390 543 kgr. for 1 m. width of arch, l = length of rolling surface = 70 cm. and k = allowable unit working stress = 240 atm. for cast iron under slow motion.

Then

$$r = \frac{2 \times 390\,543}{3.14 \times 240 \times 70} = 15 \text{ cm.}$$

To find the height of the bearing necessary to distribute the stress over the concrete, let

h = height of the bearing.

D = thickness of arch ring adjacent to the bearing = 106 cm.

b = width of bearing = 100 cm., but the rolling surface is only 70 cm.

N = normal pressure on the bearing for 1 m. width of arch = 390 543 kgr.

T = tangential stress on the bearing for 1 m. width of arch = 11 860 kgr.

M = maximum bending moment on the bearing.

$I = \frac{b\ h^3}{12}$ = moment of inertia of vertical section through axis of roller.

k = allowable unit stress = 500 atm. for cast iron, quiescent load.

Then

$$M = \frac{N}{D} \times \frac{D^2}{8} = \frac{N\ D}{8}; \text{ also } M = \frac{k\ I}{\frac{h}{2}} = \frac{N\ D}{8}$$

which solved for h gives

$$h = \sqrt{\frac{3\ N\ D}{4\ k\ b}} = \sqrt{\frac{3 \times 390\,543 \times 106}{4 \times 500 \times 70}} = 30 \text{ cm.}$$

The coefficient of friction between cast iron and stone is about 0.6; hence a tangential stress of $0.6\ N = 234\,300$ kls. would be required to

slide the bearing plate. The existing tangential stress is only 11 860 kgr. However, a small rib is provided on the bottom of the plate to prevent sliding during the erection of the structure.

To diminish the sliding friction between the roller surfaces of the bearings, it is intended to make the radius of the convex surface 15 cm. and that of the concave surface 16.5 cm. (not shown on the drawings), thus converting the sliding friction into rolling friction. The arc described by these rollers, for the extreme movements of the arch, is so small that the point of contact between the rolling surfaces would not be appreciably displaced, and there is absolutely no danger of unequal distribution of pressure on the casting, even were the motion to reach 1° of arc.

(*g*) *Composition of the Concrete.*—The full section of the arch ring for a distance of 2 m. from the hinges, and all outside the middle third of the arch ring; also the floor arches and the exposed surfaces of abutments and piers for a depth of 20 cm. from the surface, shall be of concrete, composed of 1 part Portland cement to 2 parts sand to 3 parts limestone. The middle third of the arch, the surfacing of the roadway under the asphalt composition and the cores of the small piers shall be of concrete, composed of 1 to 3 to 6. All other concrete shall be mixed 1 to 4 to 8 except the abutment foundations which shall be made as shown on Plate II.

(*h*) *Camber.*—The camber to be allowed in the arch ring will now be found for the condition that the bridge when completed, carrying the dead load and a live load of 200 kgr. per square meter, shall be at its true level at 0° Cent.

To realize this, the falsework must be superelevated by an amount equal to this camber plus the settlement which the former may undergo up to the time of closing the arch ring. The settlement in the falsework should be determined by actual tests made prior to construction.

After the arch ring is closed at mean temperature of, say, 24° Cent., and under no stress, it should be above its geometrical shape by the amount of the camber. Hence, the camber will be equal to the deflection at the crown caused by the dead load and the uniform live load of 200 kgr. per square meter, and a diminution in temperature of 24° Cent. below the mean. Then, under ordinary temperatures and loads, the arch will usually be above its theoretical position, which is very

desirable, as the horizontal thrust is materially increased by a diminution in the rise of the arch.

The thrusts N resulting from the assumed case of loading are found in Table No. 6, using equations (26), (27) and (36). The shortening in the successive arch sections between the crown and the abutment, for the respective values of N, are also found in Table No. 6 from equation (46). As the assumed case of loading is one of symmetry, only the half arch is treated.

The concrete to be used for the arch ring should possess an ultimate compressive strength of 220 atm. at probably the age when the bridge will be first tested. From Table No. 6 the average unit working stress, for the case of loading just above mentioned, will be seen to be 23 atm. Then the values of ε, α, E and E' can be taken from the data given under Section V (d), as follows: $\varepsilon = 0.0009$, $\alpha = 0.0000088$, $E = 295\,000$ atm., and $E' = 3\,000\,000$ atm. For $t = -24°$ Cent., $\varepsilon + \alpha\, t = 0.00111$ and $\frac{1}{E} + \frac{1}{E'} = 0.00000372$, which values are used in Table No. 6 to find the $\triangle L$'s.

The abutments themselves will be somewhat displaced as a result of stress and temperature effect, though the shrinkage will probably have taken place prior to closing the arch ring. Hence the equation for the shortening in the abutment may be written $\triangle L = -\frac{N L}{F}\left(\frac{1}{E} + \frac{1}{E'}\right) - \alpha\, t\, L$. Taking $F = 69\,000$ cm.^{2} as an average value, and $L = 1\,000$ cm., $\triangle L$ becomes 0.23 cm. This displacement is considered in the graphical solution on Plate II, but not in the analytical solution here following:

From Table No. 6, the value of $\delta = \Sigma_0^{\frac{l}{2}} \triangle L = 4.521$ cm., of which 3.693 cm. is permanent and 0.828 cm. is elastic.

Also $n = \Sigma_0^{\frac{l}{2}} L = 3\,789$ cm., $f = 1\,000$ cm., and $\frac{l}{2} = 3\,600$ cm.

Hence from equation (49),

$$C_1 = \left(1 - \frac{\delta}{n}\right)\sqrt{f^2 + \frac{l^2}{4}} = 3\,731.85 \text{ cm.}$$

and from equation (50),

$$\triangle f = f - \sqrt{C_1^2 - \frac{l^2}{4}} = 16.79 \text{ cm.}$$

This agrees very closely with the value 16.65 cm., obtained from the graphical solution on Plate II.

When the displacement of the abutments, amounting to 0.23 cm., is included, the total deflection at the crown will be 17.5 cm. The deflections at any other points of the arch ring may be scaled from the diagram, Plate II.

If the falsework were absolutely rigid, the crown of the arch would require a superelevation of 17.5 cm., so that the arch, if closed at 24° Cent., and when carrying its own weight and a live load of 200 kgr. per square meter, will have a rise of 10 m. at 0° Cent. The design of the falsework is not made a part of this problem.

(*i*) *The Pressure on the Abutment Foundations.*—This is found to be 4.76 atm. (see Plate II). This, for the character of the substrata, assumed in the problem as hard clay, is in no way excessive. However, when dealing with a specific case, the pressure area may be made any desired quantity and the foundation pressure be diminished to such intensity as may seem safe for the particular case.

The fact that small settlements in the abutments of three-hinged masonry arches are not attended by any serious consequences, especially when sufficient camber was put into the arch during construction, makes it perfectly safe to exceed the pressure limits, on foundations hitherto allowed for arches without hinges, by from 50 to 100 per cent.

(*j*) *Estimate of Quantities.*—The following table contains the quantities of Portland cement concrete of the various mixtures for the different parts of the structure.

Structural parts.	Quantities of Concrete, in Cubic Meters.				
	1:2:3	1:2.5:4	1:3:6	1:4:8	1:4:8:3st.
Arch ring	978	.....	478	.....	.....
Floor piers	165	.....	52	.....	.....
Floor arches and floor	328	.....	.....	.....	.....
Two abutments and wing walls	141	.....	.....	1 661	.....
Two abutment foundations	.....	60	210	.....	1 664
Totals in cubic meters	1 612	60	749	1 661	1 664
Totals in cubic yards	2 108.5	78.5	979.7	2 172.6	2 176.5

ESTIMATED QUANTITIES.

Items.	Quantities.	
	Metric units	U. S. units.
Portland cement concrete, mixed, 1c. : 2 s. : 3 broken stone...	1 612 m³.	2108.5 cu. yds.
" " " " 1 : 2.5 : 4 " " ...	60 "	78.5 "
" " " " 1 : 3 : 6 " " ...	749 "	979.7 "
" " " " 1 : 4 : 8 " " ...	1 661 "	2172.6 "
" " " " 1 : 4 : 8 b. s., 3 stone....	1 664 "	2176.5 "
Louisville cement concrete 1 : 6 : 12 broken stone..	720 "	941.8 "
Earth excavation	5 400 "	7063.0 "
Asphalt pavement over abutments, 15 cm. concrete found...	176 m².	210.3 sq. yds.
Asphalt composition 6 cm. thick over bridge	600 "	717.0 "
Concrete footwalks over abutments	88 "	105.2 "
Concrete balustrade over abutments	44 m.	144.3 ft.
Iron hand railing over bridge	150 "	492.0 "
194 m. of steel grooved rail, 80 lbs. per yd. or 39.76 kgr. per m.	7 713 kgr.	16 970 lbs.
972 steel rods, 2.5 cm. dia. 3.05 m. long, nuts each end at 12.4 kgr	12 052 "	26 514 "
120 m. *I* at 14.9 kgr. per m. or 10 lbs. per ft	1 788 "	3 934 "
Metal in expansion joints of roadway	5 000 "	11 000 "
Cast-iron hinged bearings	81 600 "	179 520 "
Falsework	72 lin. m.	236.2 lin. ft.

Unit prices have not been inserted in the above table because these are too much dependent on local conditions and market values. However, a liberal estimate of cost for the entire structure as designed, including 10% for engineering contingencies, would be about $96 000.

DISCUSSION.

WALTER G. BERG, M. Am. Soc. C. E.—This subject deserves attention and study on account of its admirably clear presentation by the author, and, further, because several bridges have been built after this method in Germany by engineers who stand very high in the profession. However, it should be remembered, that this is a structure of a class prevalent in European practice, in the design of which the tendency of the engineer has been to consider a structure as perfect, in an engineering sense, if the sections have been reduced to the actual theoretical minimum, independent of the question of additional cost of labor, and other features which come in. In American practice the aim is simplicity, even at a sacrifice of additional material. The speaker coincides with Mr. Goldmark in his statement, that, by the addition of a few inches to the thickness of the ring, which in itself is a very small percentage of the entire masonry in the structure, absolute safety can be obtained without going into these theoretical detailed methods of construction. Perhaps the solution of the whole question would be to take extra care in making the foundations absolutely safe, and so do away with the main argument in favor of the three-hinged masonry arch, namely, that it should be used where the foundation might settle. The speaker does not wish to take the stand that nothing can be learned from European practice; but he believes that the tendency in America will always be toward a simple structure, even with the use of more material, thus avoiding the very great refinement required in a construction of this kind. Mr. Berg.

J. B. JOHNSON, M. Am. Soc. C. E.—Engineers in America have made and are likely to continue to make some mistakes in the matter of avoiding those structures, the computations for which are difficult. Careful distinction should be made between a difficult computation and an uncertain computation. The speaker believes that the ordinary arch is, under all circumstances, an uncertain product, because of the unknown rigidity of the abutments and the amount of deformation which occurs in the arch when the centering is removed. When a crack is seen, it is known that the tension at that place has been more than the masonry would stand. How far that crack extends is not known, and the pressures are not known. Mr. Johnson.

Now, the three-hinged masonry arch is a structure as stable as any, and the speaker, having as much confidence in Portland cement as in the best kind of building stone, believes that, humanly speaking, it may be regarded as absolutely permanent. These arches may be built either of stone or of concrete, with or without steel embodied in the arch ring, and they may be adapted to almost any set of conditions.

The author has given a very clear, concise, and accurate solution of the problem. The argument that the solution of each problem is difficult, tedious, or unknown to most engineers, has no value. Sup-

Mr. Johnson. pose the problem is of necessity long and tedious to the greatest expert, to the man in whom is placed the highest confidence; suppose he spends a month in figuring the stresses in such a structure—this would be said to be a tedious computation; but what is a month's expense of one man computing an important structure, as compared with the cost of the structure? It is simply nothing at all. The important questions are: Are his figures representative of the facts after the structure has been erected? And is it a structure which is readily erected in accordance with theory? The speaker believes that the three-hinged masonry arch can be readily erected in accordance with theory, and that the theory is competent to determine the facts. If this is so, such structures should be accepted as normal, as proper, and as having many advantages which the present iron and steel structures do not possess, one of which, the advantage of permanence, no one would claim for iron and steel in the present state of ignorance as to the proper methods of preserving them from rust.

Mr. Goldmark. HENRY GOLDMARK, M. Am. Soc. C. E.—This paper is an able argument in favor of a masonry arch—more distinctly a concrete arch—with three hinges. The argument is the stronger because it is enforced by a detailed design of a bridge built according to the formulas developed in the first part of the paper. The formulas in question look much more formidable than they really are. The arch being built with three hinges, the author assumes that the reactions at the crown and the springing will pass through the hinges, and that the two halves of the arch will turn freely about them. The problem of determining the stresses in any section becomes, under these conditions, a purely statical one, without the necessity of introducing the more complex methods involving the theory of elasticity. The solution of the stresses can readily be reached by simple analytical or graphical methods which, for any particular case, present no difficulty. The apparent complexity of the author's formulas is due to the fact that he presents the problem in a very elegant general form, in order to make the solution more complete for any possible case that may arise in practice. The discussion of the best way of arriving at the deflections the speaker has found of much value.

The interest of the paper centers more particularly on the question whether a masonry arch with hinges is so much better than one without hinges as to warrant the substitution of these details for an arch of ordinary form. As to this, opinions may very well differ. Masonry arches have been used, almost from time immemorial, and the results, as the author states, have been very favorable. The burden of proof in favor of introducing a new form rests, of course, entirely with those who advocate the innovation.

The author's reasons for wishing to introduce the hinge are, first, his belief that in case the foundations settle the hinged arch would be safer, and could be used where the foundations are not considered sufficiently uniform and solid for an ordinary arch; second, that the

stresses being more determinate, there would be less chance of the arch cracking; and third, that temperature changes would be of less importance in the jointed arch. Mr. Goldmark.

To the speaker it seems that the substitution of this form needs stronger evidence than is brought forward by this paper. The introduction of a movable joint appears, in his opinion, to be radically at variance with the character of masonry construction. As a matter of fact, even in the author's careful design, the idea is not fully carried out. The flooring and spandrel filling are really continuous and would oppose great resistance to motion at the hinge in the crown. The result would be that cracks would take place in the spandrel or else that the hinge would remain fixed. Either alternative would be unfortunate. In the latter case the author's assumption as to the determinate character of the stresses would be vitiated. It is true that the hinges would confine the line of pressure within somewhat closer limits, but the arch would, after all, be an elastic and not a static arch. Ordinary prudence would, therefore, prescribe that the usual practice be followed and the stresses determined, both for the fixed and the hinged end conditions, so that the dimensions of the arch ring could be proportioned for the more unfavorable case.

The design which the author works out with great care differs in no essential respect from similar arches which have been built without hinges, nor is the section of the arch which he finally obtains very different from that of ordinary arches which have stood well in practice.

The hinged arch, if the hinge acts, will, however, have greater deflection and hence involve greater stresses, at least in some parts of the structure. It is hardly conceivable, at least in the case of a railroad bridge, that this can be anything but detrimental. It may be added that the introduction of metallic parts, which are subject to corrosion, must be considered objectionable in a masonry bridge.

The experiments of the Austrian Society of Engineers and Architects, made some five or six years ago, to which the author refers, show clearly that the ordinary hingeless arch, whether built of concrete, brick or rubble, does act very much as the elastic theory indicates. This theory is complex in its application, almost too much so for every-day practical use, except in rare cases. At the same time there need be no great difficulty in determining perfectly safe dimensions for even the largest arches. Arch estimates, such as the speaker has lately had occasion to make on several large arches, show that the total amount of masonry in the arch-ring is, after all, only a small percentage of the total masonry in the bridge. The addition of 6 ins., or 1 ft., to the arch, above the absolute minimum required, would make the bridge safe beyond peradventure, without increasing its cost appreciably. The economy that could be introduced by the use of hinges is, then, not very great, while it would be more than counterbalanced by the accompanying drawbacks.

CORRESPONDENCE.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The writer agrees with the author that masonry arch bridges should be built in preference to iron and steel bridges wherever possible, because they will last longer and cost next to nothing for maintenance.

To argue from this, however, that masonry arches of long span are only feasible or preferable when built with hinges, is a conclusion with which the writer does not agree. Masonry arches have the very valuable qualities of rigidity and durability. These qualities are impaired by hinges. Unyielding foundations, rigid abutments and careful work should not be less necessary for hinged arches than for fixed arches, which are better in all respects. For none of the hinged masonry arches already built can it truthfully be claimed that they are as rigid, or as good, or as durable, as fixed masonry arches of corresponding material. If the Cabin John Arch were hinged, it would not be the absolutely rigid bridge it is.

Hinges may give a theoretical excuse for paring down the dimensions of the arch ring and abutments, and for building on "any moderately good foundation"; but such bridges will not stand the test of time with good proportioned and well-built fixed masonry arches. If the foundations are not secure, and cannot be made secure, no arch bridge of any kind should be attempted.

With the same proportion of rise (10 m.) to span (72 m.) as in the author's design (page 58), and for the same moving loads and same unit pressure, equal to $\frac{1}{12}$ of the crushing load on the arch ring, a span of 140 m. (459 ft.) is readily feasible. If the proportion of rise to span is $\frac{1}{4}$, a span of 600 ft. becomes feasible under the same conditions, but in all cases the writer would prefer a fixed arch to one with hinges.

The supposed greater accuracy of calculation of hinged arches, as compared with the ordinary fixed masonry arch, is largely illusory. The fact which is not illusory, however, is that the hinged bridge is less rigid than the fixed arch bridge. The question is not that it is rigid enough, or that it is more rigid than a metal bridge. Of rigidity in a bridge there cannot be too much; if the fixed arch bridge is more rigid than the hinged arch bridge, which it certainly is, that should be sufficient reason for avoiding hinges.

But there are other, if minor, considerations: The hinges, having to be of metal, require to be looked after and kept in good condition by cleaning and painting, thus introducing the trouble and cost of maintenance, entirely absent in the fixed stone arch bridge. Corrosion of the metal hinges is not preventable, and the durability of the bridge is limited by that of the hinges.

The refinement of having the concave surface of the hinge of some- Mr. Lindenthal.
what larger radius than the convex surface, for the purpose of minimizing the friction, still more increases the liability to corrosion. Dust and rust will lodge in the hinge, where any amount of cleaning cannot remove them. Changes in the form of the arch, from whatever cause, will be localized at the hinges. Bending strains in the arch-ring are then unavoidable, and it is not possible to determine them with any definiteness. In the fixed arch, such bending strains are distributed, and, therefore, minimized, over its entire length.

The calculation of fixed masonry arch bridges presents no greater difficulties than that of hinged arch bridges.

The theory for the calculation of fixed masonry arches, based on the elasticity of the material, which the author (page 33), claims as the only one entitled to confidence, has not received general acceptance, and does not deserve it. In so far as any elasticity at all is ascertainable in building stone and mortar, it is so uneven, and between such very wide limits, that any theoretical results based on it must necessarily be equally unreliable and divergent. Elasticity has no bearing on the determination of the camber to meet the flattening of the arch after completion, which is an old arithmetical operation with data from experience on the compressibility of different kinds of masonry and foundations.

The stability and strength of stone arches of any form can reliably be determined upon the same principle which was used by the old masters who built the great German cathedrals of the middle ages. The finely dimensioned and exquisitely balanced arches and vaults, meeting on top of slender columns, were not the product of chance; they were determined by calculation, in which the "Bauwage" (a literal translation of which is building balance) was the principal instrument. It was simply a string or cord representing the center line of the arch. The string was loaded and shaped with weights representing the loads to be carried by the arch. One end of the string passed over a pulley and was balanced by a weight, which represented the horizontal thrust. The inverted equilibrium polygon, of course, corresponded in all respects to an erect one. Thus the position, intensity and direction of the thrust were known at every point. This principle, also used by Rankine as a basis for his theory of masonry arches, reaches back to antiquity, it being claimed that the cathedral builders derived it from Arabian mathematicians. It can be used with perfect safety, aided by the greater accuracy of modern analysis and graphical statics, for the largest masonry arch spans of any form.

The arch, assumed as carrying all the loads, vertically, of course, and not radially, is by this method treated as a bent wall, held in position and equilibrium between unyielding abutments. For such an arch, considered as an inert mass, the pressure line, as to form and

Mr. Lindenthal.

location, can be ascertained with an accuracy sufficient for all practical purposes; as also can the variations in the pressure line from moving load. Rankine's rule, which is an excellent one, and should be adhered to in all cases, is that the pressure line and variations of same shall not pass out of the middle third of the arch-ring section, which should be thoroughly bonded. In flat arches this condition can always be complied with. For long spans the arch material may be arranged in the form of deep ribs.

Rigid spandrels, which can very well consist of open gallery work in longer spans, good cement and good bonding, in an arch, give additional security. Piers or walls are likewise, as a rule, so proportioned that the pressure line does not pass out of the middle third. The safety of all such masonry is assured, even if the coursed stones were laid without mortar, the latter merely insuring a good fit. Hinges for fixing the pressure line through the center line of the arch are not necessary; but rigid abutments, painstaking care and good judgment in the execution of the work are of greater importance, whether there are hinges or not.

As a rule, it is much easier to design and calculate a masonry arch than to execute it as a perfect work.

The great amount of bad work in arch building and in the foundations is as responsible for cracks and failures as bad designing. The effort and skill of the engineer should be directed toward getting not merely "moderately good foundations," which the author claims are good enough for hinged arches, but to get unyielding foundations. In that respect the better knowledge of physical conditions, the better materials, and the greater resources of modern engineering, can be used with greater advantage than was ever possible before; where these cannot be had, metal truss bridges are preferable.

Brick arches, made up of separate rings, each a single brick in thickness (instead of the arch ring being bonded through like a wall), with from four to eight such rings above, and unconnected with, each other, may be seen carrying railroad tracks. The rigidity of such an arch is derived only from the spandrel filling. Yet because such arches sometimes hold up (the bricks, as it were, having more sense than the builder that put them there), that cheap practice is frequently pointed out as a good example. It is bad work of this and similar kind that has discredited masonry arches.

For long span masonry arches, but always of the fixed kind, the writer would prefer granite of the best quality, rather than the best concrete, the stones to reach through the entire thickness of the arch ring in every third course at least. The flattening of the arch, when released from the staging, is thereby distributed over all the many radial joints. But if monolithic arch masonry of the quality proposed by the author is used, greater safety against cracking can

be had, in the judgment of the writer, by embedding laterally connected iron ribs, corresponding to the Melan construction. This system lends itself very well to deep ribs in long spans. That hinges cannot be used for skew arches is too obvious for argument. Mr. Lindenthal.

The writer disagrees with the author also in the advocacy of pressures as low as $\frac{1}{6}$ of the crushing strength of the arch material. Whether concrete, or granite, or other stone of high quality, their homogeneity, with all possible care, can never be so reliably secured as to risk a factor of safety equal to that in steel or wrought iron. One-sixth of the breaking strength would be low, even for cast iron, but for stone masonry of the best kind there is no justification for less than $\frac{1}{10}$, and it should be nearer to $\frac{1}{20}$.

The durability of stone bridges is to be ascribed to their high factor of safety, necessary as a margin against weathering, and against accidental loading in the form of shock in a material, which, for greater safety, should always resist only by compression and never by tension, and principally to the great proportion of mass to moving load.

Reduce this mass in the arches and in the foundations one-half or two-thirds, as advocated by the author, under the pretense of more accurate calculation where the coarseness and variation of the material naturally do not admit of it, and stone bridges will cease to be the synonyms of strength and durability. Hinges in masonry arches are not only an incongruity, but may without prejudice be considered as one of the fads from which even an exact science like engineering is not free, or, rather, as an application of science to a wholly useless, if not injurious, purpose.

David A. Molitor, M. Am. Soc. C. E.—The opinion frequently advanced, that the design of structures of the kind under consideration requires great refinement, and also that their theoretical treatment is more difficult and cumbersome than the methods commonly used in the analysis of fixed arches, is entirely unfounded. In fact, if a fixed arch were to receive a detailed analysis, such as is given in the the paper on hinged arches, this might be justly called complicated. Yet any construction, the design of which has not received such analysis, certainly does not deserve professional sanction. It is far more important to solve a problem with exactness, even at a somewhat greater expenditure of labor, than to risk failures of construction. Mr. Molitor.

The time is fast approaching when the wasteful American methods, variously referred to, will receive the severe condemnation to which they are generally entitled, and the "rule of thumb" engineer will probably share the fate of retirement.

A careful consideration of the paper will undoubtedly convince the reader that the method is not nearly so complicated as might at first

Mr Molitor. appear, and that the analysis of a fixed arch would be far more tedious if considered in a more general manner with live loads as they actually occur. The commonly applied graphical solution is simple merely because the difficult complications are not dealt with, but supplanted by vague and often erroneous assumptions.

Another fact which has not been considered by the advocates of superfluous material, intended to cover the factor of ignorance, is that an arch of excessive thickness, and consequent great rigidity, is far more liable to crack as a result of shrinkage of the material and settling when centers are released than is a well-designed arch, with just sufficient material to carry the loads. Every arch is subject to large distortions during construction, and, unless provision is made therefor, cracks will show soon after completion.

Among the many large fixed masonry arches which the author has seen, both in Europe and America, not excluding those of his own design and construction, he can recall only a few which, in spite of the best foundations, did not show cracks. This does not mean that such arches are unsafe or that they will not be lasting, but such defects always reflect injuriously on the reputation of the designer and builder, which is unpleasant, to say the least.

In no instance has the author asserted that it is impossible to construct long-span masonry arches without hinges, but he has advocated hinges for them simply because they are more susceptible to cracks, etc., than short-span arches.

Many are the schemes which have been advanced to overcome the injurious effects of settlement during construction, but most, if not all, of them are open to severe criticism. The introduction of hinged connections has offered the best solution of the difficulty.

According to some of the criticism offered, it would be improper to construct a masonry arch on any but bed-rock foundations, which would naturally make this style of structure one of rare occurrence. To build an arch on any moderately good foundation requires the highest engineering skill and experience, while any one familiar with the use of mason's tools might safely risk the construction of an arch on rock foundations.

The question of rigidity and deflections has been strongly emphasized, but this objection to hinged masonry arches is entirely erroneous, since in them the proportion of live to dead loads is so small that the introduction of hinges does not materially influence the rigidity, as far as the purpose of the structure is concerned. The argument applies only to metal arches where the imposed loads are excessive.

In advocating low unit working stresses for masonry and concrete, the author naturally referred to a structure, the stresses in which are fully determinable. To conclude therefrom that this would be per-

missible for a fixed arch where the stresses may readily exceed the computed values by 100%, or more, is perfectly ridiculous. Mr. Molitor.

Skew arches may be built according to the method outlined in the paper, provided the skew is very slight, otherwise internal stresses arising from the skew would be likely to exert a harmful influence.

The preservation of iron hinges is not a very serious objection to the use of this style of structure; since the entire hinge may be embedded in asphaltum or some hard fatty mixture which will in no wise interfere with the freedom of action of the hinge within necessary limits, but at the same time will prevent corrosion of the metal.

The impression seems to prevail that iron is the only material suitable for such hinges. This is not true. On the contrary, such hinges can be made of a hard stone, like granite, or lead joints may be used to advantage. The choice of the kind of material rests entirely with the designer, and in the example submitted the author used cast iron.

While thanking Mr. Goldmark for his complimentary remarks, it is to be regretted that he did not give the paper sufficient study to familiarize himself with the design, instead of accusing the author of having designed a hinged arch with fixed spandrel filling. Mr. Goldmark also states that the design submitted "differs in no essential respect from similar arches which have been built without hinges," etc. No argument, as to the essential differences will carry more conviction than a careful inspection of the drawings.

The points brought out by Mr. Lindenthal have, for the most part, received consideration in the above, and it is not deemed necessary to draw out this discussion for the purpose of refuting what appears to be a recitation of text book information and personal views and preferences not necessarily accepted by others.

The examples of hinged arches, cited in the first part of the paper, certainly outweigh most of the arguments and doubts expressed by those who have taken up this discussion.

It is not in making designs and estimates or in the study of text books that the great advantages of three-hinged masonry arches may be learned and appreciated, but the constructor who has sought in vain to produce a perfect masonry bridge without a defect or blemish, will know without doubt or much argument which style of structure is best suited to practical application.

In closing, the author expresses his warm appreciation of the able support received from so high an authority as Professor J. B. Johnson.

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No. 835.

ORIGIN OF THE GULF STREAM AND CIRCULATION OF WATERS IN THE GULF OF MEXICO, WITH SPECIAL REFERENCE TO THE EFFECT ON JETTY CONSTRUCTION.

By N. B. SWEITZER, Jr., Jun. Am. Soc. C. E.

PRESENTED JUNE 15TH, 1898.

WITH DISCUSSION.

The origin of the Gulf Stream and the circulation in the Gulf of Mexico have never been satisfactorily settled. Many theories have been advanced from time to time, but none as yet have been established upon acceptable proof. The commonly received theory of the present day, that these currents flow in from the South Atlantic, pass the north shore of Yucatan, follow the coast line of Mexico and Texas in a north-northeasterly direction, and finally escape through the Straits of Florida, appears to be plainly contradicted by evidence gathered from the various surveys of the past, together with recent discoveries in connection with the several deep-water projects along the coast of Texas.

The true solution of the question seems to be that the currents coming in through the Straits of Yucatan follow one of two courses,

dependent upon the variations of barometric and planetary conditions, viz.:

(1) During the high declination of the moon, coincident with a continued low barometer off Cape Hatteras and a high barometer in the Gulf of Mexico, they flow from the Yucatan Channel in a northeasterly direction around the extreme west coast of Cuba and pass out into the Atlantic through the Straits of Florida, where they become known as the Gulf Stream.

(2) Under opposite atmospheric conditions, and during the low declination of the moon, the Channel of Yucatan pours its waters into the Gulf, so that they spread out in all directions, moving on its center; thence, being deflected by the outward flow of contrary currents, they also pass through the Straits of Florida as the Gulf Stream.

In order to understand these currents aright, especially those in the western part of the Gulf, it will be necessary to review, as briefly as possible, past observations and discoveries concerning the Gulf Stream and equatorial currents. The discovery of America may be said to be due primarily to the Gulf Stream. Long before the time of Columbus, there had been noticed in the débris constantly washed upon the shores of Scotland, Norway and France, pieces of carved wood, and canoes of curious design, quite foreign to the people of the known world. Little was needed, therefore, to convince a reflective mind, like that of the great explorer, that these things came from lands beyond the seas, which could be reached by sailing westward.

As far as known, Columbus was the first navigator to observe the oceanic currents; having noticed, when sounding in the Sargasso Sea, that the lead appeared to recede from the ship, which he rightly interpreted as meaning that the ship drifted away from the lead. Speaking of the strong currents which, on his later voyages, he noticed in the Caribbean Sea, as also among the Antilles and off the coast of Honduras, he says:

"When I left the Dragon's Mouth I found the sea ran so strongly to the westward that between the hour of Mass, when I weighed anchor, and the hour of Complines, I made sixty-five leagues of 4 miles each with gentle winds." He further adds: "I hold it for certain that the waters of the sea move from east to west, with the sky, and that in passing this track they hold a more rapid course and have thus carried away larger tracks of land, and that from hence have resulted the great number of islands."

It was about this time that Sebastian Cabot noted the Labrador current. However, the first authentic mention of the Gulf Stream is found in the log of Antonio De Alaminos, the pilot of Ponce de Leon in his famous expedition for the Fountain of Life in 1513. Having set out from Porto Rico he crossed the stream in the neighborhood of Cape Canaveral, and, after reaching latitude 30° north, he turned and coasted as far as Tortugas, stemming the current for several hundred miles. The log says that they met currents which they were unable to stem even when they had good winds; and though they appeared to be going through the water at a high rate of speed they recognized that they were really drifting backward, and that the stream was stronger than the wind. It states further that when these vessels came to anchor near the coast, one of them, a brig, being in water too deep to anchor, was soon carried out of sight by the stream.

This knowledge subsequently proved of great value to Alaminos when Cortez, in his expedition for the conquest of Mexico, placed him in command of the entire fleet. Later, when it became necessary for Cortez to send envoys to Spain to save his life, he gave Alaminos the speediest vessel of the fleet, with the instructions to sail to the north of Cuba and into the Atlantic through the Florida Straits, thus utilizing the Gulf Stream. This route subsequently became the chief course of navigation between the West Indies and Europe, and played an important part in the later development of the City of Havana.

Sir Humphrey Gilbert in some of his writings seeks to trace the motion of the waters from the African coast to America, and writes:

"The current runs all along the eastern coast of that country northward as far as Cape Freddo, being the farthest known place of the same continent towards the north," from which he concludes that "it must either flow around the north of America into the South Sea, or it must needs strike over upon the coast of Iceland, Norway and Finland."

He accepted the former theory because he was anxious to prove the existence of the northwest passage.

One of the earliest theories advanced to account for the Gulf Stream appears in "La Cosmographie," in which it is claimed that the currents in the Straits of Florida are caused by the rivers emptying into the Gulf of Mexico. John White, Governor of Roanoke, referring to a voyage from Florida Keys to Virginia about 1590, says:

"We lost sight of the coast and stood to sea for to gain the help of the currents, which runneth much swifter farre off than in sight of the coast, for from the Cape of Florida all along the shore are none but eddie currents setting to the south and southwest."

Thus it may be seen that at this early date the existence of a swift, powerful current was known; that it extended from Florida Keys, "out of sight of the coast," northward beyond Virginia; and, that between this current and the coast were "eddie currents setting to the south and southwest." In other words, the axis of the Gulf Stream had been located and the contrary current flowing southward adjacent to the coast noted.

Isaac Vassius writes, about the year 1663:

"With the general equatorial current the waters run towards Brazil along Guyana and enter the Gulf of Mexico, and from thence turning obliquely, they pass rapidly through the Straits of Bahama. On the one side they bathe the coast of Florida and Virginia and the entire shore of North America, and on the other they run directly east until they reach opposite shores of Europe and Africa."

Here it is seen that the writer notices the entrance of the currents into the Gulf of Mexico, from the direction of the Guyana coast, through the Caribbean Sea, coming into the Gulf between Cuba and Yucatan.

On their entrance into the Gulf he points out that they increase in rapidity, "turning obliquely" towards the Straits of Florida or Bahama. Strange as it may seem, these observations of Vassius, made as long ago as 1663, are much nearer the truth than those of writers as late even as to-day.

It is interesting to compare this information with the article in the "Encyclopædia Britannica" on the Atlantic Ocean (see Fig. 1). Here it is stated that the current

"passes westwards along the northern coast of South America until it is deflected northwards by the coast line of Central America, and is driven between the peninsula of Yucatan and the western extremity of Cuba into the Gulf of Mexico, at the rate of from 30 to 60 miles per day. A portion of it passes direct to the northeast along the northern shore of Cuba; but by far the larger part sweeps round the Gulf, following the course of its coast line, and approaches the coast of Cuba from the northwest as a broad deep stream of no great velocity, seldom running more than 30 miles per day."

In the light of recent investigation, the theory of Vassius appears to have been more nearly right. From the time of Vassius very little

notice seems to have been taken of the Gulf Stream until about the year 1770 when some American whalers, sailing from the Bahama banks, southeast to the Azores and north to Baffin's Bay, made the discovery that the whales stayed north or south of a certain line which afterward proved to be the Gulf Stream.

This knowledge was communicated to American navigators who found it of great value in their voyages to Europe. About this time Benjamin Franklin took up the matter and published a map which was rejected by the British Government and the English ship captains, who, true to the inherited conservatism of their nature, continued their old course, often arriving at New York when American vessels, which had left England at the same time, were half way across the Atlantic on their return voyage. Upon the outbreak of the war with England, Dr. Franklin suppressed this map, but the knowledge gained from the experience of the whalers proved to be of the greatest possible advantage to the infant navy of the American Colonies.

FIG. 1.

Dr. Franklin now made a number of voyages across the Atlantic, and from this time dates the first truly systematic and scientific investigation of the Gulf Stream. After noting several observations of the temperature made with the aid of the thermometer, Franklin says:

"I find that it (the Gulf Stream) is always warmer than the sea on each side of it, and it will appear from thence that the thermometer may be a useful instrument to the navigator; since currents coming from the northern into the southern seas will probably be found colder than the water of those seas, as the currents from the southern seas are apt to be warmer."

A. von Humboldt next published a remarkable work on the Gulf Stream, in which he maintained that it was not the same at all seasons, but depended to a great extent upon the wind. This is found to be true, at least as far as velocity is concerned.

The excerpt on page 91 is taken from an exhaustive report by Lieutenant Pillsbury, U. S. N.:

"During the first quarter of this century the British Admiralty office had collected a great quantity of material on the subject of ocean currents and meteorology, most of which had never been known to the public. Mr. James Rennell, who had devoted his life to the subject of geography and particularly to ocean currents, was given the task of compiling and collecting data. He combined the results on large charts of the ocean, which were the admiration of the day. He also wrote a volume entitled, 'An Investigation of the Subjects of the Currents in the Atlantic Ocean.'

"Major Rennell adopted Dr. Franklin's theory as to the principal cause of ocean currents, and divided them into two classes, drift currents, caused by constant or long-continued winds on the surface of the water, and stream currents which are formed by the accumulation of water by the drift currents meeting an obstacle and thrown sideways or out of its usual course. The Gulf Steam he placed in the latter class. He considered the water in the North Atlantic a drift current impelled by the prevailing westerly winds, and these also were the cause of the African current."

From these and other observations Major Rennell draws the following conclusions:

(1) "That there existed a change in the position of the column of warm water from time to time"; (2) "that the breadth varies at times in the proportion of more than two to one"; (3) "that these changes had been observed sometimes to be very sudden, *e. g.*, on one occasion the stream had been found to be 140 miles in width, and in two weeks later at the same spot it was 320 miles;" (4) "that these changes did not follow any regular course of the seasons, for it was 320 miles wide in May, 1820, and only 186 miles in May, 1821, nearly at the same place;" (5) "that on the northern side of the stream the body of warm water is more permanent than on the south, and that the warmest water is found to the north, as if indicating the strongest part of the stream there;" (6) "that the existence of warm water does not necessarily indicate the presence of the stream, but must be regarded as an overflow or deposit of superabundant water; or even from a counter current;" (7) "that there are without doubt veins of colder water within the body of warm currents."

These deductions of Major Rennell were a vast advance upon anything that had previously been attempted. True, some of them are faulty, but it must be remembered that the temperature observations were those of the surface, and that surface-water, impelled by a gale of wind, will traverse many hundreds of miles in a short space of time. Besides this, his conclusions were gathered from totally unconnected data, such as reports of merchantmen, naval officers, and ship captains

of all classes, whose observations, especially for longitude, were often careless in method and detail. A period of about twenty years elapsed before anything further was accomplished in the exploration of the Gulf Stream. Then, in the year 1844, Professor A. W. Bache, of the United States Coast Survey, began the first really scientific and comprehensive survey of this great ocean river.

Origin of Ocean Currents.—It will be necessary to notice some of the more important theories advanced. Columbus held the opinion that the waters followed the motions of the heavens about the earth, in which view Sir Humphrey Gilbert concurred. Kepler believed the flow of the currents to be considerably influenced by the motion of the earth; the cohesion of the particles of water being less than that of land, the water was naturally left behind in the revolution of the earth on its axis.

The scientific world at that time was considerably agitated by the opposing theories of Passine and Furnier. The former held that the currents were caused by the heat of the sun attracting the ocean, forming an immense mountain of water, which vessels had great difficulty in ascending. He thought that this mountain moved constantly westward until it met the South American coast, which turned it and caused it to move northwestward. Furnier, on the contrary, claimed that the sun caused an immense hole to be made by evaporation into which the waters rushed, thus causing the currents. Like the fable of the two knights on opposite sides of the statue, they were both right to a certain extent.

Kircher, a learned mathematician of Würzburg, attributed the currents to the effect of the winds, which, when deflected by the shore line, formed currents now designated as stream currents. He also noticed the influence of the moon on these currents.

Dr. Franklin strongly advocated the theory that the constant winds blowing toward South America caused a rising in the level of the water, which, being deflected mostly northward, entered the "Bay of Mexico," flowing north of Cuba, and from thence to the Banks of Newfoundland. Von Humboldt attributed the Gulf Stream to the winds, the melting of ice in the polar regions, and the revolution of the earth on its axis.

Lieutenant Maury, U. S. N., in his valuable work entitled "Physical Geography of the Sea," says that the ocean currents are due to

various causes, the more important of which are: *first*, the difference in the specific gravity of sea water in the tropics and in the polar regions; *second*, the influence of the winds. He says:

"Difference of specific gravity is the *chief* cause. Whenever the waters in one part of the sea differ in specific gravity from the waters in another part, no matter from what cause the difference may arise, or how great may be the distance between two such parts of the sea, the heavier water will flow (by the shortest route) towards the lighter; and the lighter in its turn will seek the place from whence the heavier came."

"In other words, from whatever part of the sea a current flows, back to that part a current of equal volume must flow."

He then introduces two qualifying theories to explain the western flow of the equatorial current, viz., the rotation of the earth and the winds. There is no doubt that the evaporation in mid-Atlantic, in the latitude of Cape St. Roque, between the continents of Africa and South America, far exceeds the precipitation, and consequently the specific gravity there must be greater than that of either the Arctic or Antarctic Oceans, which are little more than brackish.

If, therefore, Lieut. Maury is correct in his supposition, that the difference of specific gravity is the chief cause of oceanic circulation, then there would necessarily be a flow of the heavier saline waters from the equator direct to the south and north, and a consequent return of the lighter and fresher waters from the polar regions.

Such, however, is not the case, and to account for the flow of ocean currents another cause than that of specific gravity must be sought. This cause will be found in the winds which blow constantly from Africa to South America and the West Indies. The equatorial current moves directly west, unaffected by its specific gravity. The rotation of the earth doubtless does affect the ocean currents, but its effect is far less than that of the winds. Specific gravity too may be a considerable aid to oceanic circulation, but it is a mistake to suppose it to be the prime cause. Difference of barometric pressure, especially in shallow bays and gulfs, perceptibly affects the currents by accelerating or retarding their velocity. The effect of this force is seen to be very great when it is remembered that a difference of 1 in. in the barometer will be accompanied by a difference in the elevation of the Gulf of more than a foot. Taking into consideration the vast quantity of water involved in such elevation, one can realize something of the tremendous volume which pours into the Atlantic.

Circulation in the Gulf of Mexico.—Having reviewed the history of the Gulf Stream investigation and the more important of the theories advanced to account for its origin, the original question, concerning the course followed by the currents between the time of their entrance into the Gulf of Mexico at Yucatan, and their departure through the Straits of Florida, will be considered.

The question has never been regarded as satisfactorily answered, and it is a remarkable fact that even eminent authorities like Maury and Pillsbury seem to evade it. Maury says :

"It (the current) enters the Caribbean Sea and the Gulf of Mexico, from whence it issues through the Straits of Florida as the well-known Gulf Stream."

Lieutenant Pillsbury says in his report, page 591 :

"We have thus followed the water driven by the *vis a tergo* of the trade winds from the coast of Africa to the Yucatan Channel, from which it flows into the Gulf of Mexico and through the Straits of Florida into the Atlantic."

Reference has already been made to the popular opinion, as expressed in the "Encyclopædia Britannica," viz., that the waters come in through the Yucatan Channel and follow the coast line of Mexico and Texas, finally departing through the Florida Straits; but a more reasonable suggestion is found in the report of the Superintendent of the United States Coast Survey published in 1895, in which he states that the waters of the Gulf of Mexico are "erratic in direction, and feeble in force," and suggests what appears to be the correct theory, that the directions of the currents entering and leaving the Gulf are dependent upon the declination of the moon and certain conditions of barometric pressure.

The winds can have but little influence over the waters in passing through these channels, as they are deep-streamed currents, flowing at a probable depth of from 1 000 to 1 500 fathoms; but the circulation in the Gulf is unmistakably dependent to a large extent upon the winds.

It was once thought by many that the Mississippi River was the "fountain-head of the Gulf Stream " and caused a circular motion in the Gulf by flowing direct from its mouth to the Straits of Florida. The absurdity of this idea is readily seen from the fact that the amount of water emptied into the Gulf by this river is only 0.36 cubic mile per twenty-four hours,* while the amount received through the

* Humphreys and Abbot.

Yucatan Channel is 652 cubic miles in the same length of time. This fancy is still further disposed of by the discovery that the current of the Mississippi enters the western part of the Gulf and flows in a southwesterly direction.

Here is a good illustration of the opportunity afforded of tracing a stream of fresh water into the sea by means of the difference of density; for by this means the waters of the Mississippi are easily traceable from its mouth in a southwesterly direction, to a point only a little south of the latitude of the Rio Grande, and not far east of the longitude of Galveston.

The specific gravity theory will now be considered. The specific gravity map of the Gulf * shows that the line of maximum density extends from about 300 miles north of Yucatan nearly west to the eastern shores of Mexico. The northern half of the Gulf is quite light, except off the western coast of Florida. If, then, this specific gravity theory be the right one, there will be a flow from the east, southeast and south toward the north and northwest. This, to a certain extent, is true, especially where the motion of the waters is accelerated by the prevailing winds which produce drift currents along the northern and western shores of the Gulf. For nearly ten months in the year these winds blow constantly from the south to a little north of east and for at least nine months they seldom ever change from the southeast. During these months it is not only constant, but strong, and causes the waters to move toward the northwest. When they reach the northern part of the Gulf they are deflected by the shore line, and move westward. On the other hand, the waters from the southwestern part impinge on the coast of Mexico, and produce the current which flows north along the shores of Mexico and Texas.

These two currents meet in the western part of the Gulf, raising the surface and thereby producing a return current which passes south and eastward toward the Straits of Florida. The current along the coast of Mexico is feeble in force near its starting point, off the west of the Yucatan peninsula, but gradually increases until it reaches its maximum velocity between Tampico and the Rio Grande, where it flows at the rate of 3 miles an hour, and then continues to diminish until it meets the contrary current from the east.

The circulation of the waters in the Gulf is shown in Fig. 2.

* Report of the United States Coast Survey for 1895, p. 369.

A considerable agitation of the waters, covering an area of about 100 square miles, occurs off the west coast of Texas, about 40 miles south and 20 miles east of Aransas Pass, which can only be accounted for as resulting from the meeting of two opposing currents. In the immediate neighborhood of this phenomenon the coast is covered with débris of every description, among which the fir tree of Wisconsin, the palmetto of Florida, the cocoanut and other products peculiar to the tropical regions of the South are found lying side by side.

The chief obstacle to harbor improvements along the coast of Texas is the vast quantity of sand which, being stirred up and held in suspension by the waves, is carried by the currents and deposited in the channels of harbors. A current, to carry heavy sands, must have a bottom velocity of about 2 ft. per second, thus necessitating a much greater surface velocity, which it is known these currents do not have, of themselves, under ordinary circumstances, as they rarely move faster than $2\frac{1}{2}$ miles per hour. Therefore, it is concluded that the wave action must be the chief factor in the dislodgement of the sands, and, in connection with the littoral currents, determines the position and form of harbor jetties.

FIG. 2.

The author's attention was first attracted to the littoral currents by the construction of a north jetty at Aransas Pass, Texas, in 1895. Prior to that time attempts had been made to obtain deep water by the construction of jetties on the south side of the channel, but they failed through the lack of capital required for their completion.

In that year the Aransas Harbor Company built a jetty in the form of a letter S, on the north side of the channel, running east and west half a mile from the southeast extremity of St. Joseph Island. This work was conducted upon the supposition that the littoral current came from the north, and that it would be so deflected by the jetty as to enter Aransas Bay between the western end of the jetty and St.

Joseph Island. It was further imagined that during ebb tide this current would meet the outgoing current in the Pass, thus neutralizing both velocities and preventing the outward flow from passing north of the jetty; or, if the ebb current was the stronger, they would pass out through the channel on the south side of the jetty.

Long before this work was completed the author became satisfied that the littoral current came from the south, as, by a series of experiments, he had previously proved to be the case at Pass Cavallo, not far north of Aransas Pass, where the conditions were the same.

When the jetty was completed it was found that there was less water in the channel than before the work began. Colonel Goodyear, the last contractor, recognized the mistake that had been made, and proceeded to finish the jetty on the south side of the Pass, thus excluding the drift sand brought in by the currents and waves from the south, with the gratifying result that the author's latest survey, made in June, 1897, showed a marked increase in the depth of the channel. Further evidence is unnecessary to establish the fact that, at this point, at least, the littoral currents flow from the south.

If the jetty construction upon the coasts of Texas and Mexico is examined, it will be found that these currents maintain a uniformly northern course from Tampico to Galveston, where they meet those coming from the westward from the mouth of the Mississippi.

At Tampico the south jetty was built first in order to protect the channel from southern sand-bearing currents. At Brazos, Santiago, Texas, the same thing was done. Lieutenant G. A. Zimm,* writes:

"The direction of the channel across the bar depends upon the direction of the winds and littoral currents. During nine months of the year southerly winds blow and there is a littoral current from the south."

The same thing is noticed on the sketch accompanying Captain McClellan's report on this harbor in 1853. At Pass Cabal only one jetty was built, and that was also placed on the south side. A jetty was commenced at the mouth of the Brazos River on the north side, but Major Ernst, in his report of September 6th, 1887, remarks:

"The map of recent survey shows the channel, instead of running southeast in the direction intended, turned off at a right angle and running northeast across the jetty."

* Annual Report of Chief of Engineers, U. S. A., page 1330.

At Galveston the South Jetty was built first. At passes east of Galveston, however, the jetties are almost invariably constructed on the east side, to check the sand-bearing currents in their westward flow. Among these may be mentioned Calcasieu River Pass, La.;* also Sabine River Pass.†

It is well known that at the mouth of the Mississippi the littoral currents flow from the east. Hence, it may be concluded that the currents flowing westward are on the northern shore of the Gulf, and those flowing north along the western shore meet at some point near Galveston, the location of which is variable, as it is dependent upon the direction and force of the winds.

Summing up these observations of the Gulf currents, it is found that there is abundant evidence of the presence of two sets of currents in the Gulf of Mexico, viz., deep-stream currents and littoral-drift currents.

The former enter the Gulf through the Yucatan Channel, and, under certain barometric and planetary conditions, pass by the western extremity of Cuba and flow out through the Straits of Florida, or else, under converse planetary and atmospheric conditions, they spread out over the Gulf in all directions, moving on its center.

The littoral-drift currents are originated through the agency of the prevailing southeast winds and flow northward along the western boundary of the Gulf, and west along the northern boundary, meeting in the vicinity of Galveston and forming a stream-current which flows in a southeasterly direction toward the Straits of Florida.

In the past there has been far too little attention paid to the motion of the Gulf waters. Were they better understood, there can be no doubt that vast sums of public and private capital might have been expended more judiciously than they have been, resulting in more real and lasting good to the people of the coast country. If, therefore, this paper is the means of inducing a greater interest in these Gulf currents, which have such a direct and important bearing on trade and commerce, the author will feel that it has successfully accomplished its mission.

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* Report of Major W. H. Heuer, 1886.

† Report of Secretary of War for 1895.

DISCUSSION.

L. M. HAUPT, M. Am. Soc. C. E.—The title of the paper is broad, and the research shown in presenting the subject is commendable. It is true that the engineer should ascertain the remote as well as the immediate causes operating to produce certain observed effects, in order that he may properly apply them to the wants of man. Mr. Haupt.

It is also true that unless a correct diagnosis is made of the effects and their causes, one may readily fall into errors in reaching conclusions, and *errare est humanum*.

Turning first to the general theory advanced by the author, it becomes necessary to refer to the physical features of the gulf littoral to interpret his description correctly.

The author, in the opening of his paper, contravenes the "commonly received theory," that the currents "flowing in from the South Atlantic pass the north shore of Yucatan, follow the coast line of Mexico and Texas in a northwesterly direction, and finally escape through the Straits of Florida."

In his subsequent exposition of his own theory the author admits the correctness of the direction of ingress and egress, leaving in dispute the disposition of the currents in the basin of the Gulf when he says that under certain conditions they flow (1) northeasterly around the west coast of Cuba and pass off directly into the Straits, or (2) under opposite conditions "they spread out in all directions, moving on its center; thence, being deflected by the outward flow of contrary currents, they also pass through the Straits of Florida as the Gulf Stream."

This leaves the matter in rather a chaotic condition as to the origin of the counter deflecting currents and the direction of the resultant littoral movements.

Passing the history of the origin of the Gulf Stream, and turning to page 95, the author states with reference to the specific gravity theory: "If, then, this specific gravity theory be the right one, there will be a flow from the east, southeast and south toward the north and northwest. This to a certain extent is true, especially where the motion of the waters is accelerated by the prevailing winds," which are said to prevail for nine months in the year. "They seldom ever change from the southeast. During these months, it is not only constant, but strong, and causes the waters to move toward the northwest." This leaves us in a quandary. If that is the resultant direction and flow for nine months of the year, the question naturally arises as to the disposition of all this water, and why is Texas not inundated? But this is explained as follows: "When they reach the northern part of the Gulf they are deflected by the shore line, and move westward. On the other hand, the waters from the southwestern part impinge on the

Mr. Haupt. coast of Mexico, and produce a current which flows north along the coast of Mexico and Texas." That is on the portion of the coast south of Tampico. These two currents, as he explains, meet in the western part of the Gulf, raising the surface, and produce a return current, passing south and east toward the coast of Florida. Consequently there must be two conflicting currents moving in nearly opposite directions, one of them nine months in the year, and the other effecting its escape across the path of the first in some unexplained manner. No attention is paid apparently to the vertical movement of the currents, which is a very important factor in all these studies. As an illustration of that, attention is directed to the Narrows in New York Bay, where the maximum movement is apparently outward; whereas, the inward or flood movement prevails in the bottom of that gorge for eleven hours out of the twelve. There is a vertical eddy at that point, and material deposited in the Lower Bay is carried to the Upper.

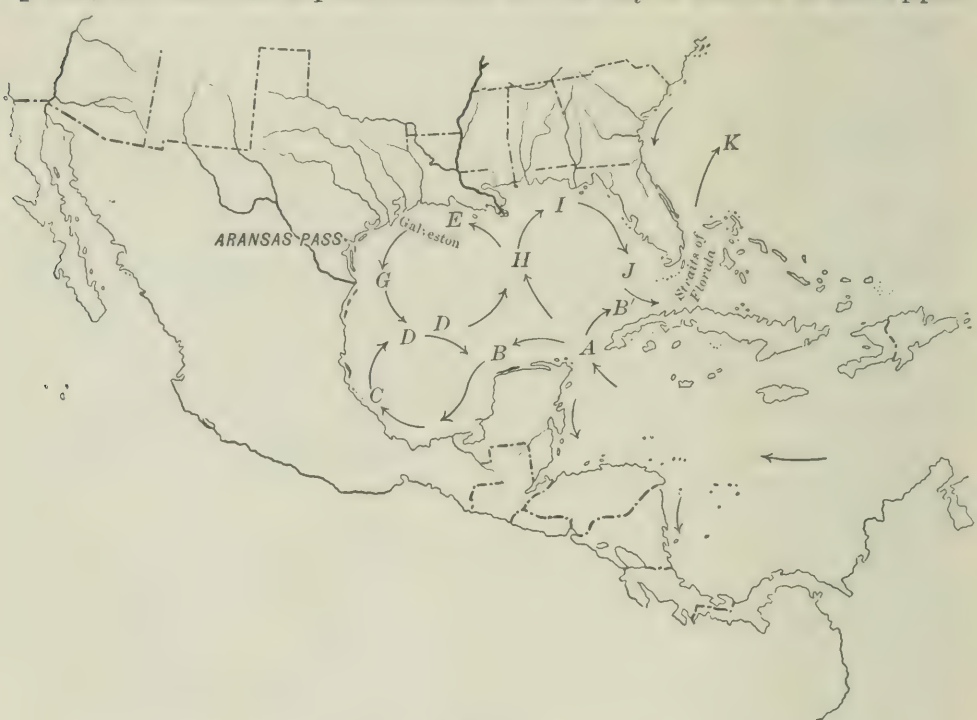


FIG. 3.

The author continues: "The current along the coast of Mexico is feeble in force near its starting point, off the west of the Yucatan peninsula, but gradually increases until it reaches its maximum velocity between Tampico and the Rio Grande, where it flows at the rate of 3 miles an hour." But what becomes of it? The diagram (Fig. 2) shows it to be reflected toward the center of the Gulf, where it is met by the entering currents off Yucatan.

These were some of the thoughts suggested by a perusal of the paper. They leave the matter in an unsatisfactory condition, but it

is an important as well as an interesting question from a theoretic and scientific standpoint. Mr. Haupt.

It may be *àpropos* to suggest a theory of circulation which may fit the case. Let it be assumed that the impelling stream enters the Gulf as a rack would enter between two gears or pinions. As it comes in, the currents are dispersed to the right and left into these lateral basins, where, being diverted by the configuration of the coast, eddies will be formed of greater or lesser magnitude, the one to the right, the other to the left, while the movement of the water in the isolated or protected bay west of Yucatan will take up a reverse direction, so that the currents from this overflow, one hugging the northwest shore, the other the southwest, will form a resultant uniting again with the main stream, and this action seems to conform very closely to the observed results. In other words, under this theory, there should be three large eddies in the Gulf which indicate the direction of the flow of the currents as shown by the letters *A B C D — E G D — A H I J — A B′ K* (Fig. 3).

Coming to the practical applications, which, of course, are the more important factors for engineers, a map published in the reports of the Chief of Engineers shows the peculiar feature, that at all of these Texas entrances, the discharge through the ebb channel or the main ship channel lies in the direction of the meridian, that is, nearly due north and south. This is very suggestive because it shows that as the western bight of the bay is approached, the channel is found to lie nearer the western shore. The position of the channel with reference to the inlet is an important characteristic. It is fully discussed in the paper on "Physical Phenomena of Harbor Entrances."* In a case of that kind the indications would be that it is the resultant of a littoral movement, taking the seasons through for the entire year, and the ebb currents would be flexed to the south, while the bar crossing would be at the point of least resistance. That being the case it furnishes the key to the solution of these harbor problems, and the development recently made, or proposed to be made, in this science is to create the depths by the single jetty instead of two, as formerly.

The continental practice has been to attempt to utilize the ebb currents by concentration between two jetties, so that the currents may scour out between them, but it has not been found satisfactory in the long experience of maritime engineers. It invariably happens that wherever two jetties of that kind are built, they have to be extended as the groins connecting them with the shore fill with sand.

A more recent examination, made August 2d, by H. C. Ripley, M. Am. Soc. C. E., and the writer, at a season when the southeast winds tend to build up the bar, showed a remarkable deepening and a progressive and rapid improvement in the channel, there being over 18 ft.

* *Proceedings*, American Philosophical Society, 1888.

Mr. Haupt

at points where there were formerly but 8 ft. The pilots report deeper water on the bar than they have ever known to exist.

These facts would seem to refute the statement of the author that a mistake had been made and that the results of the work were injurious.

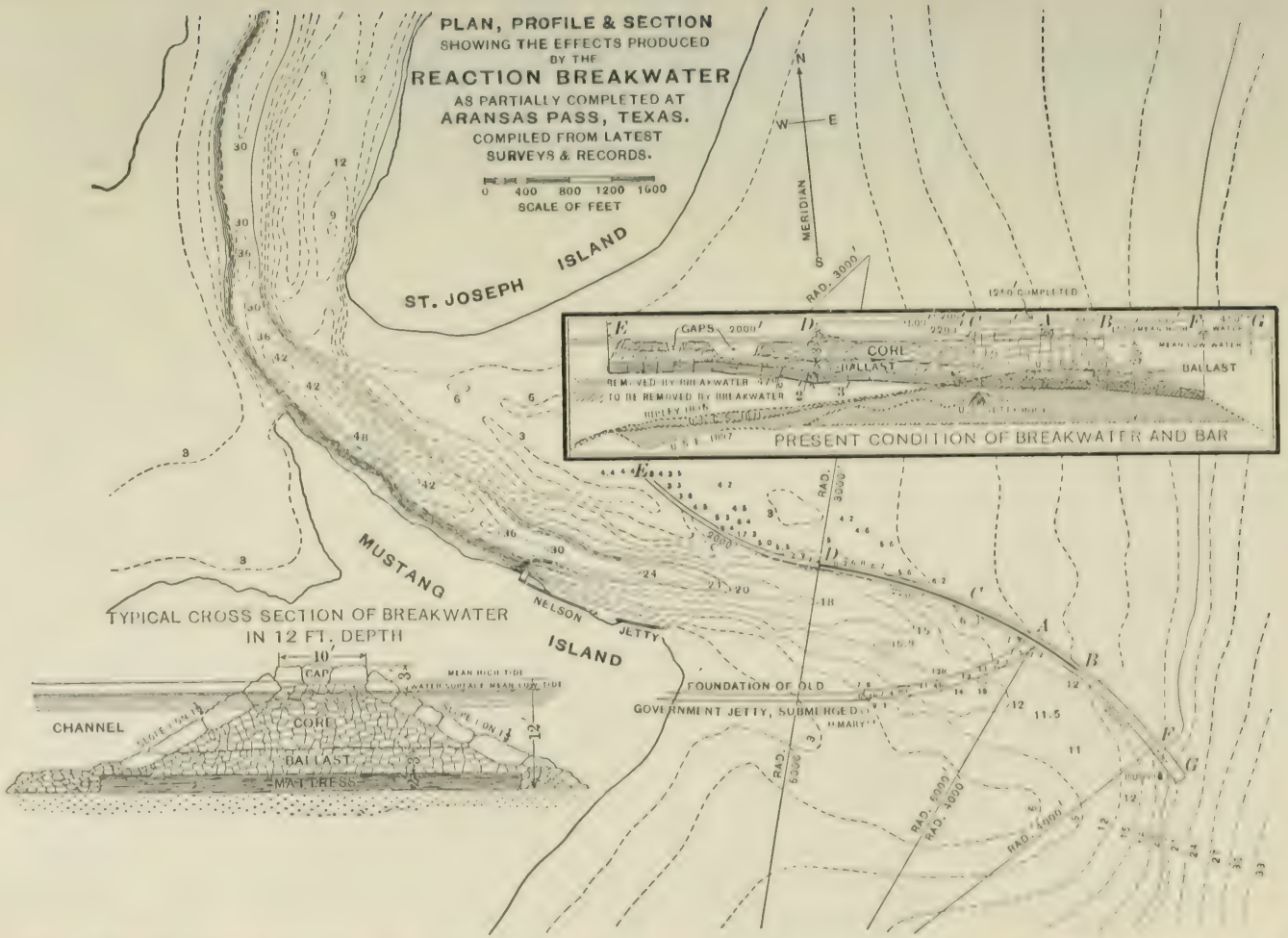
There are a few points in the paper which the speaker is unable to reconcile with the well-known and accepted phenomena at the Gulf inlets, and which are fully confirmed by the Reports of the United States Engineers and the comparisons of the Coast Survey charts. This confusion may arise from a failure to make a clear distinction between the off-shore and littoral currents, which would seem to have resultant motions in opposite directions; but, however this may be, it is generally assumed that the motion of a body is in the direction of the resultant of the component forces acting upon it. This being a physical law, the direction of movement of the inlets (when unrestricted) should indicate the direction of the forces causing that effect. Hence, when it is found that an inlet is moving to the south, it is difficult to understand how the littoral currents (producing that progression) flow from the south. For example, the case cited, Aransas Pass, has unquestionably moved over a mile, in 25 years, to the south, the average rate being stated as 260 ft. per annum. This being a fact, it is difficult to understand how the impelling force can flow from the south.

The author makes several statements as to the *S*-shaped jetty, which, he remarks, was built "on the supposition that the littoral current came from the north," etc. As the originator of this design and one of the consulting engineers, who has been kept well informed as to progress and results, the speaker regrets to have to take exception to the accuracy of the statements which his colleague has thus publicly made, and desires to make the necessary corrections, *seriatim*.

The author states :

"Long before this work was completed the author became satisfied that the littoral current came from the south, as * * * at Pass Cavallo, * * * where the conditions were the same."

As this work is still far from being completed, it would seem unjust to deduce conclusions therefrom, inasmuch as the breakwater was not designed to change the direction of the littoral drift, from north to south, and has not done so, but to utilize its agencies; first, to arrest the sand and defend the channel from its encroachments; while, secondly, it applied the changed conditions of equilibrium, which it created in the currents, to cause a rapid erosion at ebb, by reaction, across the crest of the bar. So remarkable, indeed, were these results that in less than three months about half of the bar was removed by natural scour, without any advance seaward, and an increase of depth of over 4 ft. was secured, and all from a structure not then half com-



ONLY THE PORTION B C WAS COMPLETED. THE REST OF THE BREAKWATER E G IS IN AN UNFINISHED CONDITION, YET THE BAR HAS BEEN ABOUT HALF REMOVED WITHOUT ADVANCING SEAWARD. THE SUBMERGED GOVERNMENT JETTY PREVENTS FURTHER DEEPENING OVER ITS CREST, WHERE ROCK IS FOUND AT 13 FT.

FIG. 4.

Mr. Haupt.

Mr. Haupt. pleted. Such a result is unprecedented in the annals of harbor-bar removal by natural agencies.

It may excite curiosity, therefore, to discover wherein lies the truth of the statements contained in the next paragraph, as follows:

"When the jetty was completed it was found that there was less water in the channel than before the work began. Colonel Goodyear, the last contractor, recognized the mistake that had been made, and proceeded to finish the jetty on the south side of the Pass, thus excluding the drift sand brought in by the currents and waves from the south, with the gratifying result that the author's latest survey, made in June, 1897,* showed a marked increase in the depth of the channel. Further evidence is unnecessary to establish the fact that, at this point, at least, the littoral currents flow from the south."

This would seem to point out the danger of generalizing from special cases, for, while the sand does drift at certain periods of the year, to a limited extent, from the south, its resultant direction is from the opposite quarter. As to the correction of "the mistake" by the completion of the south jetty by Colonel Goodyear, there is no room for great differences of opinion. If this south jetty was completed, it was not platted on the surveys and was never reported to the consulting engineers, who were merely advised that some sand-bag revetments were placed on the south bar to temporarily concentrate the ebb currents on the real cause of the difficulty, viz., the old Government jetty covered with rock, and the curved portion of which was reported to have "practically disappeared" seven years before this work was located, but which, unfortunately for the people of Texas, was discovered to be in place lying directly across the channel at depths varying from "8 to 14 ft.," and which no current could remove. It was found to act as an effectual retaining wall and barrier to the acquisition of the requisite depths, and the plan has been officially condemned because the 15 ft. predicted for this half-finished portion was not secured, but without recognition of the fact that this obstruction, placed in the channel by former engineers and reported to have disappeared, still remains in place and is the cause of the failure to secure 15 ft. by scour.

As already remarked, the jetty was never completed and the outer end of the channel is unprotected (see Figs. 4 and 5), so that the bar formation at and beyond this old jetty is not under the control of the currents, and consequently this portion of the bar remains with but very little material to be removed, and with variable depths. The present low-water depth is 11 ft., covering only a ship's length, while there are holes extending to 15 ft. on both sides of the obstructing jetty.

If the author made a survey in June, 1897, and found "gratifying results," he must have been cognizant of the fact that prior to that

*On the Company's chart of June, 1897, Fig. 5, no such jetty is shown.

date the contractor had exploded some 23 000 lbs. of dynamite on the old government jetty, in his effort to remove it, and that, in consequence of the partial breach made, the inner contours advanced seaward and remained permanently in this new position; but, as the contour maps show only a space of about 300 ft. in width over this jetty, having depths of "13 ft. and rock," it was not sufficient to permit the currents to do their work effectively; yet the author attributes the benefit solely to the sand bags placed near the old south (Nelson) jetty. Is this a full diagnosis of this problem, and is the conclusion justifiable? It seems remarkable, first, that, in the recent reviews of this incomplete work, no stress has been laid upon the existence of the old

Mr. Haupt.

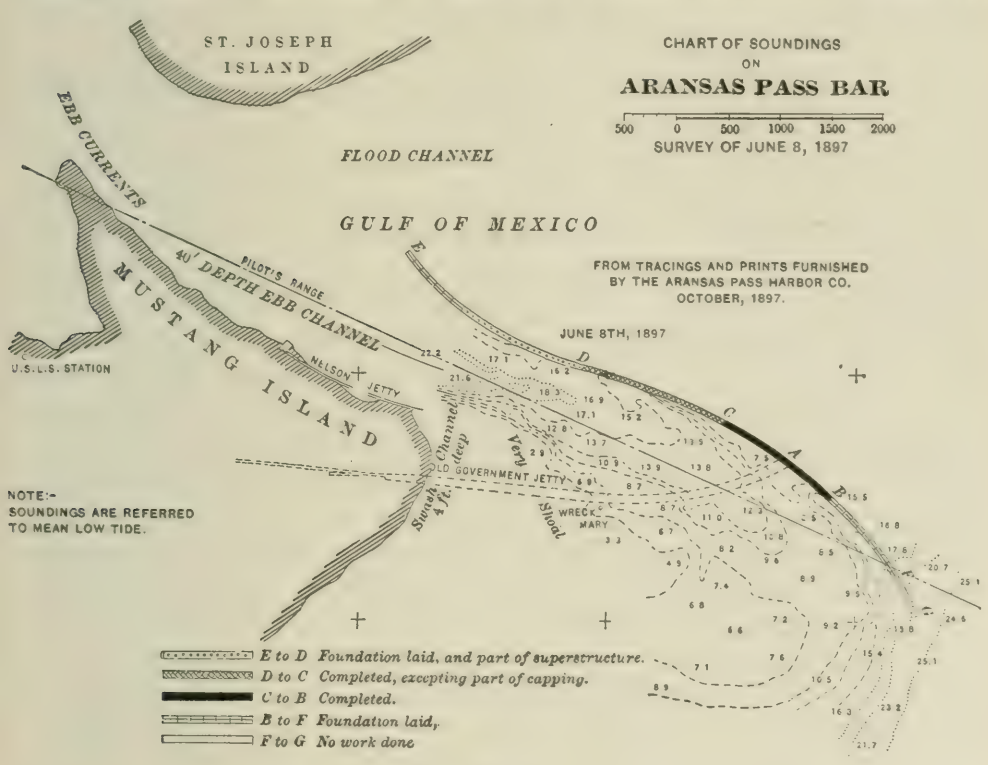

ST. JOSEPH ISLAND
CHART OF SOUNDINGS ON ARANSAS PASS BAR
500 0 500 1000 1500 2000
SURVEY OF JUNE 8, 1897
EBB CURRENTS
FLOOD CHANNEL
GULF OF MEXICO
FROM TRACINGS AND PRINTS FURNISHED BY THE ARANSAS PASS HARBOR CO. OCTOBER, 1897.
JUNE 8TH, 1897
PILOT'S RANGE
40' DEPTH EBB CHANNEL
MUSTANG ISLAND
U.S.L.S. STATION
NELSON JETTY
OLD GOVERNMENT JETTY
WRECK MARY
Shoal
NOTE:- SOUNDINGS ARE REFERRED TO MEAN LOW TIDE.
E to D Foundation laid, and part of superstructure.
D to C Completed, excepting part of capping.
C to B Completed.
B to F Foundation laid,
F to G No work done

FIG. 5.

"Mansfield" rock jetty sunk directly across the channel and reported to have disappeared, yet which has proven to be the main cause of the interruption to the excellent results which the first part of the breakwater was producing; second, that the jetty is regarded as "completed," which is far from the fact; third, that the depths after such alleged completion were found to be less than before the work began, and fourth, that after the alleged completion of the "south" jetty by Colonel Goodyear (which was never built in the sense indicated, and which is not shown on the survey of "June, 1897," or any other), it should have produced such "gratifying results," and to show such a marked increase in the depth of the channel as to require

Mr. Haupt. no further evidence to establish the author's premises. The inventor of this system invites discussion, but desires that it be fair, open and honest, and that due weight be given to all the factors *pro* and *con*, in order that the relative value of this method of improving ocean bars may be fully revealed. If far greater results can be secured by one jetty than by two, it merits a fair test, which it has not yet secured. Aransas Pass is an admittedly difficult location, yet it is claimed that this partially built breakwater has produced, at that point, incomparably greater results than those secured by the government, at one third the cost and in one-twentieth the time. Ten years have elapsed since it was first made public, and the various respectful and courteous requests, made of the parties having jurisdiction over harbor works, to permit a test to be made, have been silently ignored.

It so happened that at Aransas Pass the private parties who held a franchise from the Government, having failed to secure results from other devices, concluded to try this plan, but for financial reasons they determined to handicap it at the start by cutting it in twain, and agreed to build only "1 250 ft. of complete breakwater and 2 600 ft. of apron extension," covering the portion *F B C D* upon a predicted result of 15 ft. Such a result would undoubtedly have been secured but for the fortuitous circumstance of the old jetty lying at about 13 ft. below the surface, and covered with rock which was reported to have been scoured bare, and which the Company and contractor neglected to remove, for reasons of their own. This much of history seems necessary to explain some of the misrepresentations as to the results of this design and to show its remarkable utility and economy as a method of improving harbor entrances, and in developing the commercial resources of the country, in which the profession should take a deep interest at this time.

The experience at Aransas Pass points to the belief that the resultant littoral drift is, therefore, not from the south on this portion of the Texas coast, as the author states, but from the north. This is sufficiently indicated by the prevailing directions of movement characterizing the inlets along this portion of the Gulf littoral.

Mr. Pitts. Thomas D. Pitts, Jun. Am. Soc. C. E.—As the author refers to the results of the work at Aransas Pass, as evidence in favor of his theory, it seems proper that the speaker, who was for two years assistant engineer on that work, should make some statement as to the conditions and results there, especially as he is informed that the late chief engineer, Wm. Dunbar Jenkins, M. Am. Soc. C. E., will be unable to contribute to the discussion.

The speaker does not desire to discuss the author's general theory, as he is not sufficiently familiar with the subject, and that theory has already been so ably discussed by others, but he would like to say that the author seems to base his theory largely on the prevalence over

the Gulf, for nine months of the year, of a strong southeasterly wind; Mr. Pitts.
and that, while this wind does prevail at points on the coast where records have been kept for a term of years, as at Corpus Christi and Galveston, the pilot charts of the United States Hydrographic Office show the average yearly wind resultant as being about east by south in the western half of the Gulf and nearly east in the eastern half.

On page 97 the author refers to the jetty at Aransas Pass as being completed, but to this statement exception must be taken.

The jetty is now only half completed, and the portion yet unfinished is a most vital part of the work, since its completion would fully control the tidal volume, only a part of which is now utilized to produce scour.

It does not seem quite fair to conclude, because a half completed work fails to do what the whole work was expected to accomplish, either that the work is a failure or that the theories and observations on which the plans were based were erroneous.

When work on the jetty at Aransas Pass was begun in July, 1895, the entrance was closed by a crescent-shaped bar, with channel depths varying from 9 or 9½ ft. in winter to 8½ ft. or less in summer. The line of the proposed jetty crossed the crest of the bar near the point *A* (Fig. 5), and the axis of the channel in use at that time about midway between the points *C* and *D*. The depth in the channel was about 9 ft., and the least depth on the axis of the present channel about 8 or 8½ ft.

The consulting engineers had predicted that the completion of the portion *B C* of the jetty, and the laying of the foundation 3 ft. high above the bottom from *C* to *D*, and from *B* to *F*, would give a navigable depth of 15 ft., it being understood that the jetty must be completed at once in order to hold this depth. It was known that the Government had laid the foundation of a south jetty, about 10 years before, which, if still in existence, would form a dam across the proposed channel, but it was believed* that this foundation had entirely disappeared.

At first the results were very gratifying, and depths of 13 ft. and greater were obtained in a few months, together with a very marked reduction in the width of the bar, but it soon developed that the old Government jetty had not disappeared, and that the 13-ft. channel passed through a low spot or break in this jetty, the average depth over the rock of the old jetty being from 9 to 12 ft., from the wreck of the *Mary* to the outer end. Of course, rock having been reached, it was physically impossible for the current to scour a deeper channel.

On March 1st, 1896, the condition of the jetty was as follows (see Fig. 5): A foundation 1 000 ft. long and 3 ft. high above the bottom was laid from *F* to *B*, the depth of water over the rock being from 8 to 13 ft.; from *B* to *C*, 1 250 ft., the jetty was fully completed; from *C*

* On the authority of the Report of the Chief of Engineers, U. S. Army, for 1888.

Mr. Pitts. to *D*, 1 500 ft., the foundation was laid, and a riprap mound had been built thereon to the level of mean low tide, and had then been allowed to wash down and take its natural slope under wave action, the depth of water over the rock being from 4 to 6 ft.; and a foundation 3 ft. high had been laid for a distance of 1 400 ft. from *D* toward *E*. At this time there was little or no current across the bar, the ebb flow passing out between *C* and St. Joseph Island, and most of it at and just west of *D*. About this time, too, was the beginning of the season of southeasterly winds in that year, and they blew almost without a stop during the months of March, April, May and part of June. No one who has lived on that part of the Texas coast can doubt that during the prevalence of these southeast winds there is a large sand movement along shore to the northward and eastward. It would be hard to devise a better trap to catch this sand and cause it to be deposited on the bar than the jetty in its unfinished condition, and the depth on the bar did, in fact, at once begin to decrease.

Work was carried on, when the weather permitted, during the four months mentioned, but was suspended, for want of funds, from the latter part of June until the first of August. At the time it stopped, in June, the portion *C D* had been built to the water surface and partly capped, but the greater part of the ebb flow and the strongest currents still passed out between *D* and St. Joseph Island.

After much urging, the Company provided a limited amount of money, with which work was resumed early in August. By the end of September, when the funds were again exhausted, the foundation had been extended to *E*, and a narrow ridge of rip-rap had been built to the level of mean low tide between *D* and *E*, except in two places, one about 400 ft. long, near the middle of *D E*, and one about 100 ft. long at a distance of 600 ft. east of *E*, where gaps were left, the money on hand not being sufficient to close them. The depth of water over the rock in the smaller gap is about 6 or 7 ft., and in the larger one 5 or 6 ft., and the current through them approaches $4\frac{1}{2}$ miles per hour during ebb-tide. No work has been done on the jetty since September, 1896, and it is to-day in substantially the condition described above.

On October 1st, 1896, the depth on the bar was not more than $6\frac{1}{2}$ ft., and during October and November it decreased to 6 ft. As stated above, when work was resumed in August, there was but a feeble current across the bar, but as soon as the portion *D E* was partially completed to low-tide level, as described, it increased to a surface velocity of 3 ft. per second and more.

About the first of December, work was begun under the contract with Colonel Goodyear, and was actively prosecuted until May. During this time 23 500 lbs. of dynamite were exploded in the channel, and the "south jetty," mentioned by the author, was built. This latter

was composed of sand bags with a little rock protection on the channel side. It did not at any time extend more than 300 ft. from shore, and has since been washed away. The speaker believes that this jetty had no appreciable effect upon the depth of water on the bar. The dynamite, however, had a good effect, especially by breaking up and partially removing about 300 ft. of the old Government jetty. Mr. Pitts

In February, 1897, the depth on the bar was $7\frac{3}{4}$ ft., and in June it had increased to $8\frac{3}{4}$ ft. In November last the depth on the bar was a little over 9 ft., and when the speaker left Aransas Pass in March it was 10 or $10\frac{1}{4}$ ft. He is informed that it has continued to increase since that time, and that the depth on the bar is now 11 ft., notwithstanding the southeast winds and northerly currents of the past spring.

A part of the increase between December, 1896, and June, 1897, was doubtless due to the use of the dynamite, but the speaker does not think that all, or even the greater part, of it came from that cause, for there was every reason to expect an increase in depth during the winter, which is the season of strongest ebbs; and, besides, the depth has continued to increase slowly since the use of the dynamite ceased, thus showing the good effect of the breach in the old Government jetty and the partial completion of *C D E*.

The speaker wishes to emphasize the fact that before the portion *D E* of the jetty was partially built up to low-water level, the currents were not controlled, and did not cross the bar with any strength, and that it was before that section was built up as described that the shoaling, cited by the author in support of his theory, took place.

Since June, 1897, a shoal has been noticed lying to the south of the channel, the shape and position of which are shown by the contours on the map. During the past fall and winter this shoal has been dry at extreme low tide for a distance of 1 000 ft. or more from shore, except for a narrow and shallow channel close to the beach, and it was possible to wade out to the wreck of the *Mary*, where in July, 1895, there were depths of 12 to 15 ft. This shoaling seems to be due chiefly to an eddy which is formed to the southward of the channel during ebb-tide, and which causes the sediment in a portion of the ebb waters to be deposited, as well as part of that carried by the littoral current. The eddy seems more marked when the littoral current is flowing to the southward than when it flows in the opposite direction, but in either case sand is deposited on this shoal. There has also been a marked shoaling to the north of the jetty, covering a greater area, but of less depth. This result was expected to follow the building of the jetty, and indicates a littoral current from the north, because there is no eddy here to account for the shoal as there is on the south side of the channel.

In conclusion, it seems hardly correct to call this work a failure, when, in spite of delays and partial completion, it has radically

Mr. Pitts. changed the location of the channel and fixed it in position, has caused an increase in depth of 5 ft. or more from the worst condition of the bar, which condition itself was caused by failure to properly push the work, as urged by the engineers; has given an available depth of 2 ft. more than could be found when the work began, and has caused the large advances of the inner deep-water contours. All this was accomplished when only half of the total amount of material required had been put in place, and without causing any advance seaward of the deep-water contours on the outer face of the bar. The crest of the bar is now a little outside of *B*, and the speaker believes that this would soon disappear if the section *B F* were completed and the old Government jetty entirely removed.

Instead of considering this work a failure, the speaker thinks that it has not yet had a fair trial, and that the results have been remarkable, in view of all the unfavorable conditions. He believes that a depth of 15 ft. would have been obtained during the winter of 1895–96, as predicted by the consulting engineers, but for the fact that the foundation of the old Government jetty lay directly across the channel, at a depth of about 11 ft., and prevented the current from cutting deeper. No provision was made by the Harbor Company for the removal of this foundation, although they were repeatedly urged to do so by the engineers.

It is quite likely that a spur jetty on the south side of the channel would have been found desirable to keep out the sand-laden northerly current during the spring and summer months, and so prevent fluctuations in the channel depth; but there is no doubt that the jetty, as originally designed, would have secured the depths it was planned to secure, had it been pushed to completion in a reasonable time, provided the old Government jetty foundation had been removed as soon as it was discovered.

The jetty at Aransas Pass as yet gives no conclusive evidence regarding the direction of the littoral currents on this part of the Texas coast, but after two years' observation and study, the speaker believes that the yearly resultant of the littoral drift is south and not north. This seems to have been the cause of the former decided southerly movement of the Pass. There is, however, at certain seasons, a strong, persistent, sand-bearing current, flowing north along the shore, which must be taken into account in designing harbor works on this part of the Gulf coast.

Mr. Cameron. Brewster Cameron, Esq.—The speaker does not presume to discuss this subject from a professional standpoint, but merely wishes to state the results of his observations as a practical man. A map of Aransas Pass and the bays opening into it, made by a Mexican engineer about one hundred years ago, when Texas was a part of Mexico, shows that the pass or entrance to Aransas Harbor was then at a

point opposite Lydia Ann Islands, or about 1½ miles northerly or easterly from the present location. This map, together with the official surveys of the United States Government covering a long period of years, conclusively shows that the Pass has steadily worked its way southwesterly. This has doubtless been caused by the heavy northerly winds that prevail during the winter season. These winds drive the waters of the bays with great force in a southwesterly direction, causing a continual erosion of the south side of the Pass and a constant cutting away of the islands which are situated on the southerly side of all passes along the Texas coast. The same winds that produce these heavy currents in the bays, and which lead to this great scour, produce equally strong currents in the Gulf near the shore, and these currents carry a great amount of sand southwesterly along the Texas coast, producing a heavy littoral drift which builds up the islands on the north side of the Pass, as the heavy currents in the bay cut away the islands on the south side of the Pass, thereby maintaining the normal width of the channels to all the harbor entrances. Mr. Cameron.

These physical facts demonstrate conclusively the basic error in the paper, viz., that the drift is from the south or west. A casual investigation of the conditions that exist at Aransas Pass is very likely to be misleading. There is a light prevailing wind from the south or west during the summer, which causes a slight movement of the sand toward the north or east. It is so noticeable near the surface that it is likely to deceive a casual observer, but it is so insignificant, as compared to the drift from the north, that it is a grave error to make the statement that the resultant drift at this port is from the south. The movement of the islands in a southerly or westerly direction shows conclusively that this is not a fact.

As to the conditions that exist on the coast of Texas, the speaker's conclusions coincide entirely with the opinions expressed by Messrs. Ripley, Wisner, Haupt and Pitts. In connection with the harbors of Texas all these gentlemen have had extensive personal experience, and particularly Major Ripley, to whose exhaustive experiments, carried on through a long period of years, the people are indebted for the results published in the official reports by the United States Government. These reports have incontrovertibly established the fact that the resultant drift is from the north.

Referring particularly to the conditions existing at Aransas Pass, fortunately actual experiments have been made which amount to a mathematical demonstration of the fact that the resultant drift is from the north or east. The fact may be cited that the Mansfield jetty, which was constructed, as at Galveston, on the south side of the Pass, extended the bar seaward as the jetty was being constructed, and produced no additional depths over the bar; whereas a spur jetty only 600 ft. long, constructed by the late Colonel George W. Fulton, on the

Mr. Cameron. north side of the Pass, secured an increased depth of 2 ft. over the bar, as shown by the reports of the Corps of Engineers, U. S. A., and without extending the bar gulfward. Against demonstrated facts of this character, the mere expression of opinion by anyone should have no weight whatever. It is unfortunate that the author has fallen into these errors, as it is most important to the entire West that the favorable conditions existing at Aransas Pass should be correctly understood, and it is greatly to be regretted that he has misunderstood the whole theory of the harbor improvement contemplated by the construction of the reaction breakwater. So far from being a failure and not realizing the expectations of those who designed it, the speaker personally knows this to be erroneous. Messrs. Haupt and Ripley were employed by him to prepare the designs for the improvement of Aransas Pass, and the work, so far as constructed, has far surpassed the expectation of the speaker and the predictions of the engineers, having scoured out so great a quantity that the distance between the outer and inner 20-ft. contours has been reduced nearly half a mile. The scouring force of the current produced by the reaction breakwater, which was only partially completed, has not only accomplished this, but has rapidly deepened the bar until the work of erosion was stopped by the old Government jetty, which lies directly across the channel at a depth of from 11 to 13 ft. The sand covering this obstruction was scoured down to the bare rock, which the currents could not remove and the contractor did not, since it was not included in his contract.

Instead of being a mistake, as stated by the author, the work has been designed, located and executed with consummate skill, and every anticipation of the engineers has been realized as to intercepting the drift, regulating the currents, fixing the location of the channel and inducing rapid scour by this reaction breakwater, with the single exception of securing the predicted depth of 15 ft. to result from this incompleted work. As already stated, it failed solely because of the obstructing Government jetty, which had been officially reported as having long since disappeared. Certainly, if there is anyone to blame for failure it would seem to be the United States engineers for not correctly reporting the facts as to the existence of the old jetty, and for taking advantage of this neglect to remove the obstruction erected by them, to condemn the reaction breakwater.

The speaker believes that nowhere else in the world have such great results been secured in so short a time and at such moderate cost, and that by this method of treating ocean bars the profession of civil engineering had made a distinct advance.

CORRESPONDENCE.

GEORGE Y. WISNER, M. Am. Soc. C. E.—The author has apparently based his conclusions, as to the direction of the littoral currents along the Texas coast, on the fact that, at most of the harbor entrances, the south jetties have generally been constructed first, and that such jetties were for the purpose of shutting out sand from that direction. When it is considered that such construction has resulted in the waste of millions of dollars, and in some cases has caused a deterioration of the channels which the works were designed to improve, it may be well to examine more closely into the facts of the case. Mr. Wisner.

From the Rio Grande to the mouth of the Mississippi River there is not a single harbor entrance, where the formation is such as to show the effect of littoral currents, that does not furnish conclusive evidence that the resultant direction of currents is from the northeast.

It is true, as stated in the paper, that these shore currents are greatly influenced by the force and direction of the prevailing winds, but, from a careful examination of the weather records for that section, it will be found that, while the wind is generally from the south during the summer months, the force is so small that the currents to the northeast seldom extend much below the surface; whereas the winds from the northeast, east and southeast, during the autumn, winter and spring months produce currents of from 2 to 3 miles per hour, and to a great degree are the controlling factors causing sand movement along the coast.

At the mouth of the Mississippi River the sediment transported through the jetties is entirely carried to the westward of the entrance, causing a shoal half a mile outside of the end of the west jetty.

Previous to the improvement of South Pass, the movement of the bar seaward was 100 ft. per year, but for the past eighteen years the littoral current across the entrance to the jetty channel has prevented any advance, or the formation of any bar in front.

At the mouth of the Brazos River, Texas, the phenomenon of sediment deposit is exactly similar to that at South Pass. The Brazos is one of the heaviest sediment-bearing streams on the Gulf Coast; and, while the outer face of the bar was pushed seaward 3 000 ft. by deposits from the river during the construction of the jetties, the littoral currents completely eroded and carried away to the westward this entire deposit within a year after the works were completed to the crest of the bar.

The statement as to the unusual agitation of the waters of the Gulf, 40 miles southeast of Aransas Pass, will need to be substantiated be-

Mr. Wisner.

fore being accepted as the source of the Gulf currents through the Straits of Florida.

The direction of the resultant littoral current along the Texas coast has been established beyond question by careful observers, whose conclusions are entirely in accord with those which should be arrived at from a study of the form and movement of the natural harbor entrances on that coast.

The works at the entrance of Aransas Pass were not based on any "supposition" as to the direction of the littoral current at that place, but on known conditions; and the results already obtained from the incomplete breakwater indicate that, when completed, and the old jetty across the channel is removed, the required depth to the harbor entrance will be obtained and easily maintained.

It is well known that the flow across the outer end of the breakwater will cause an eddy to the westward of the works, which will carry sand into the channel, unless the south jetty be carried far enough seaward to prevent such action. The plans for the Aransas Pass work contemplated such construction, and, so far as the writer knows, no change has been made in this regard since the work was started.

At Galveston the action of the eddy to the west of the jetties has been sufficient to erode the shore along the city front to a considerable extent.

The utility of a single jetty for producing deep and safe entrances at harbors on sandy coasts depends largely on the direction of the littoral resultant along the coast, and the length of spur jetty needed on the lee side of the entrance will vary with almost every location.

At Aransas Pass the spur on the south side of the channel will have to be carried out some 2 000 ft. from shore to prevent the inflow of sand.

The River and Harbor Bill for 1896 appropriated $2 345 000 for the improvement of the entrance to Cumberland Sound, Ga., by means of parallel jetties. At this place the movement of sand is from north to south, and, if a properly located north jetty be constructed first, the required depth of channel would be obtained, and more than $1 000 000, required under present plans for building the useless south jetty, would be saved.

Mr. Wrotnowski.

A. F. Wrotnowski, M. Am. Soc. C. E.—This interesting subject cannot be too lightly handled, and the data for its study should be of the most authentic class in order to give it a sound basis for a proper consideration. The author, perhaps, is in a measure correct as to the general trend of the flow of the littoral currents along the Texas coast, but these currents are so varied in their direction at other points as to defy the assertion that their trends are in positive and well-defined directions. The writer has measured currents at depths of

5 ft., 10 ft., 15 ft., 20 ft. and 25 ft. in the Gulf, opposite and about two miles off the ends of the jetties, on various occasions during their construction, and in a number of instances the surface, 5-ft. and 10-ft. floats, went southward, while those of greater depths went northward against the surface current, and, generally, the middle-depth floats were undecided in their movements. On these occasions there were slight winds from the north which caused the surface current to run southward. At various other times the writer has also noticed ships anchored in the offing and heading north during a stiff southerly breeze. This, however, can be accounted for by the fact that for a few days previous the wind had been strong from the north, thus setting the current southward. The indications are that the shore or littoral currents are mainly governed by prevailing winds, and do not, as a general rule, form a material factor in determining the location for jetty works. Mr. Wrotnowski.

The author is misinformed about the South Jetty at Tampico having been built before the North Jetty. In fact, the latter was out about 1 500 ft. before the former was commenced. However, his views upon the greater accumulation of sand deposit next to a south jetty seem to be reinforced, as, in the case at Tampico, there is now much more sand south of the South Jetty than is the case north of the North Jetty. This difference, however, is due in a great measure to the fact that during northers the water accumulates and is driven against the North Jetty, and finds its way out, in an increased current, licking the North Jetty with such energy as to scour the sand from that side and sweep it past the front of the jetties to the south side.

ALEXANDER E. KASTL, M. Am. Soc. C. E.—During the four years, 1888 to 1891, inclusive, the writer was connected with the Brazos River Harbor Improvement, in Texas, as Principal Assistant Engineer and Engineer, and with the Tampico Harbor Improvement, in Mexico, as First Assistant Engineer. His observations were that the resultant direction of the littoral currents on the Gulf coast is southwestward at the mouth of the Brazos, and southward at Tampico. These currents are sand-bearing. At the mouth of the Brazos the northeastward currents produced by southwesterly winds are mere surface currents and have no appreciable effect on the sand movement. The same is true of the northward currents at Tampico. A study of the detail charts of the Brazos River and Tampico Harbors will show that the sand movement is southward. Mr. Kastl.

In reference to the jetties at Tampico the north jetty was started first, and, later on, the south jetty. Both jetties were carried seaward together. The original intention was to begin work on both jetties at the same time, but, as the railroad facilities were all on the north side of the river, earlier and greater progress was made on the north jetty.

Mr. Kastl. The writer was at Tampico Harbor when the work was started, and during his stay there, until April, 1891, the work on the north jetty was always in advance of that on the south jetty. The plan of improvement did not contemplate the building of either jetty first, but the building of both jetties at the same time, seaward, as fast as possible, and this plan was followed as nearly as it could be. Tampico Harbor is the mouth of the Panuco River, and the improvement accomplished the removal of the bar at the mouth of the river. Before the jetties were built the depth of the channel over the bar was not more than 10 ft., whereas now there is a channel of about 25 ft.

Much information in regard to the littoral currents along the Texas coast at the mouth of the Brazos River is contained in a paper,* entitled "The Brazos Harbor Improvement," by George Y. Wisner, M. Am. Soc. C. E.

Mr. Raymond. THOMAS L. RAYMOND, M. Am. Soc. C. E.—The importance of the author's conclusions in defining the limits and direction of the littoral currents of the Gulf of Mexico, can scarcely be overestimated in their practical bearing upon the methods and ultimate efficiency of the work of harbor improvement on the coast of Louisiana, Texas and Mexico. From the Mississippi River to Tampico there are only two harbors to-day which will admit vessels of greater draft than 12 ft. over the bars, though there are many which are of ample capacity inside the sand-bars blocking the entrances.

The improvement of some of these harbors, in addition to those now under treatment, is certainly the work of the near future, and any testimony corroborative of the record of facts cited by the author must prove of value in deciding upon the economical location of the controlling structures at any given place.

At Sabine Pass, to which the author refers casually, the limited appropriations for starting the work enforced the construction of only a portion of one jetty under the first contract, and it was decided to build the west jetty foundation first, on the theory that the littoral current, which was well understood to flow westward, would meet the outward current from the harbor and produce a stronger eroding stream in the direction of the resultant and parallel to the line of the jetty.

Sabine Pass is 60 miles east of Galveston and unlike most, if not all, other harbors upon the Gulf of Mexico, had not a suspicion of sand upon its bar, and only a narrow fringe of sand on its western bank.

There was much surprise when it was discovered after the foundation of the west jetty had been built, that a sand bank had formed against it covering the work completely in places, and extending, with diminishing thickness, 3 000 ft. to the eastward.†

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* *Transactions*, Vol. xxv, 1891.

† See report to Capt. Turtle, in Report of Chief of Engineers for 1885.

So improbable did it appear that this deposit had come from the littoral current, when so little sand was found in the vicinity, that its formation was attributed to the scour from the harbor above and sediment from tributary streams. Mr. Raymond.

In the light, however, of the author's investigations of the experience gained and increased depths produced by the construction of the east jetty, it seems certain that this sand was caught by the obstruction presented by the west jetty to its westward flow along the shore. The fact that this deposit was found only upon the outer slope of the bar in depths from $6\frac{1}{2}$ ft. to 9 ft. beyond the line of the Gulf shore, where the bottom had formerly been a soft clay, easily penetrated by a prod to the depth of 30 ft., adds to the probability of the conclusion that this sand was brought from the eastward by a littoral current of considerable velocity.

Had the east jetty been built first, it is fair to presume that this drift would have been excluded from the channel, and that the increase in depths would have been realized, in some degree at least, before the construction of another jetty.

Another phenomenon observable at all the mouths of the Mississippi River, at Calcasieu Pass, and, in a marked degree, at Sabine Pass, is the projection of the western banks of these harbors, further into the Gulf than the eastern shore line. At the entrance to the South Pass jetties, for instance, the shoaling beyond the end of the west jetty has built up to within 7 ft. of the surface for 1 000 ft. seaward of the terminus of the works, while the widest and deepest channel has for 16 years turned sharply to the eastward around the end of the east jetty, directly in the teeth of the undoubted westerly littoral current.

At Sabine Pass, where the outflow is practically free from sediment, the sand deposit is found only on the west bank near the entrance. This seems to indicate that the westerly drift, previously described as caught by the foundation course of the west jetty, which was built first, meeting the outward current from the harbor, is carried by the resultant current into the influence of the eddy west of the entrance and thrown up on the west shore.

The tendency is, therefore, to build out the western shoals toward the channel in a direction opposed to the set of the littoral current. When the obstruction of the west jetty foundation caught this sand drift, this effect was produced in a marked degree in one year, and the line of deepest water moved a considerable distance to the eastward.

H. C. Ripley, M. Am. Soc. C. E.—The author has opened up a subject of the greatest interest to engineers devoted to the study of problems involving the effects of ocean currents, and, while he has apparently given the subject much study as regards its literary Mr. Ripley.

Mr. Ripley. aspect, he seems to have fallen into serious error as to the local conditions on the Texas and Mexican coasts.

The writer is familiar with these conditions at all the important harbors on the Texas and Mexican coasts from Sabine Pass, Texas, to the Coatzacoalcos River, Mexico, having made personal examinations, more or less extensive, at the following places, viz.: Sabine Pass, Galveston Harbor, Brazos River, Brazos Santiago, Tampico, Vera Cruz and the mouth of the Coatzacoalcos River, and his study of the subject extends over a period of more than twenty-five years. It is the purpose of the writer to limit his discussion to those conditions of which he has personal knowledge.

It is undoubtedly true that there are times when the littoral current along the coast of Texas flows to the north and east; but it is equally true that at times it flows in the opposite direction. The writer has observed this phenomenon many times and at many places along this coast. Under his direction a series of observations, extending from 1880 to 1884, were made on Galveston Bar at such times as work was being done on the south jetty, to determine the direction and force of these currents. The result showed that the southwesterly currents largely prevail during the fall and winter months, while the northeasterly currents prevail during the spring and summer months. It has also been observed that when the wind is easterly or northerly the littoral current flows southwesterly, and when the wind is westerly or southerly the current is in the opposite direction. This has led to the conclusion that the littoral current along this coast is to a large extent, if not entirely, dependent upon the wind. The coast of Texas from Sabine Pass to Aransas Pass, is nearly straight, and has a general direction of southwest by west. The line normal to this direction is southeast by south and northwest by north. Winds from either of these directions will be directly on or off shore, and hence their effect in producing littoral currents will be *nil* unless it be considered that the water banked up by on-shore winds must find a lateral escape along the shore in either direction from some central point; and similarly the depression caused by off-shore winds may cause a tendency toward the restoration of the equilibrium by lateral shore currents. Winds blowing from directions from N. W. by N. around to the N. and E. to S. W. by S. will tend to produce a southwesterly littoral current, while winds from directions from N. W. by N. around to the W. and S. to S. W. by S. will produce a northeasterly littoral current.

In the report of the Chief of Engineers, U. S. A., for 1880, page 1 220 *et seq.*, there are thirteen diagrams which show the direction of the wind, the number of miles traveled and the duration in hours for each month in the year from June, 1879, to May, 1880, inclusive, and for the whole year, made from the records of a self-registering

anemometer at Galveston, Texas. The latter diagram (for the whole year) is reproduced in Fig. 6, and to it is added a dotted line through the center of the compass rose indicating the neutral axis of wind effect upon littoral movement. The direction of this line is N. W. by N. and S. W. by S. The full line shows the number of miles traveled by the wind, and the dotted line the corresponding time. The scale for the wind is in miles, and for the time is in hours, the circum- Mr. Ripley.

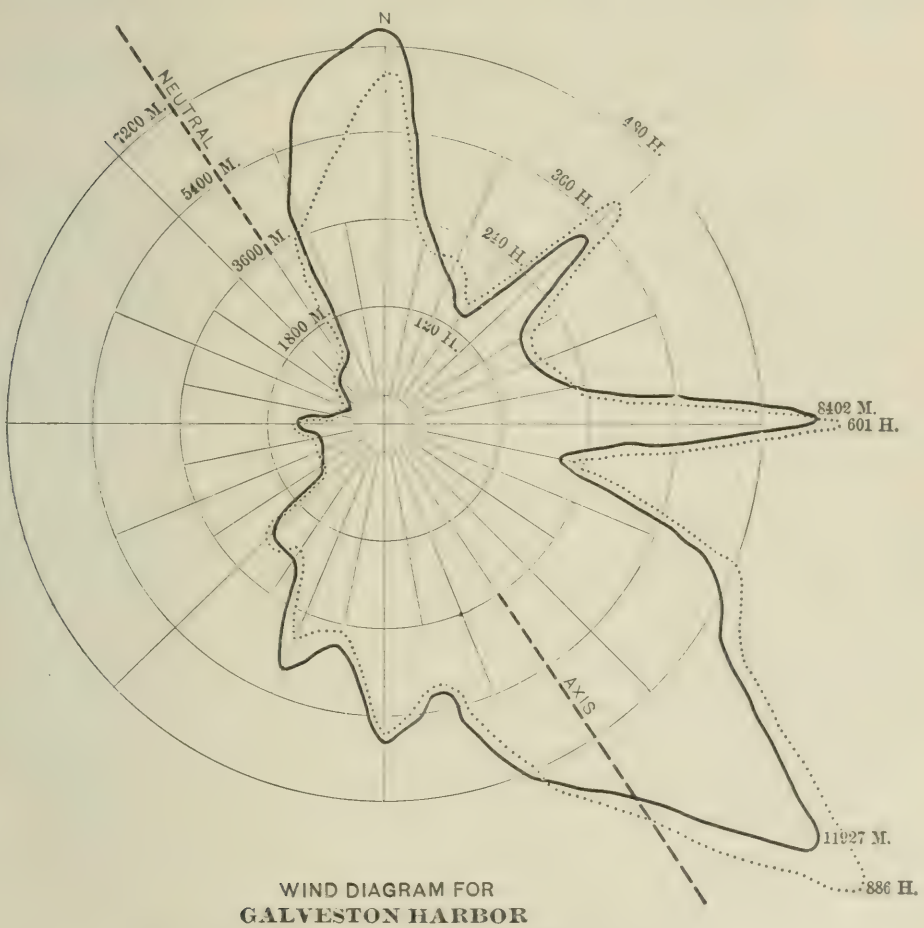

FIG. 6.

ference of the small circle at the center of the compass rose being the zero of the scale. The area embraced within the curve to the right of the neutral axis (looking N. W.) represents the amount of wind tending to produce a southwesterly littoral current, while the area embraced within the curve to the left of the line represents the amount of wind tending to produce a northeasterly littoral current. It may be seen from these diagrams that winds favorable to the production of a south-

Mr. Ripley. westerly littoral current prevail almost wholly throughout the months of September, October and November, the greater part of the months of January, February, March, May, June and December, a small part of April, and somewhat more than half of the months of May and August; while the winds favorable to the production of a northeasterly littoral current prevail during nearly the whole of the month of July, the larger part of April, a considerable part of August, and only a very small part of January, February, March, May, June and December. The diagram for the whole year (Fig. 6) indicates that more than two-thirds of the winds throughout the year are favorable to the production of a southwesterly littoral current.

However, this is not all. The littoral movement, that is, the movement of sand along the shore, is dependent upon other conditions than simply that of the current. A current may flow gently in one direction for a long time without producing any appreciable effect in sand movement, unless the sand is stirred up and held in suspension by wave action, and it is well known that those winds which tend to produce a southwesterly current also produce the heaviest breakers upon the coast; hence the littoral movement to the southwest is much greater than is represented by the duration of the littoral current in that direction.

The resultant amount of the littoral movement along the northwest and west coast of the Gulf of Mexico is more definitely shown by the character of the bars and channels at the various passes along the coast. In their original condition the channels leading from the passes across the bar to the Gulf were invariably flexed to the south, with two notable exceptions, namely, the mouth of the Brazos River, in Texas, and the mouth of the Panuco River, in Mexico. The cause of the southerly flexure at other places is the resultant southwesterly littoral movement, which deposits sand upon the northeast side of the channel and forces it over to the south. This was very evident at Galveston where the channel after passing beyond the gorge, turned abruptly to the south and extended for 4 miles before crossing the bar. At Pass Cavallo the channel hugs the west shore of the Pass for more than 4 miles to the south, when by going southeast a much shorter route to the Gulf could be had. At Aransas Pass the channel formerly turned south inside of the wreck *Mary*, crossing the bar far to the south of its present position. The easterly flexure of the channel at the mouth of the Brazos River is caused by the alluvial deposit from the river, to the west of the mouth, due to the southwesterly littoral current. This deposit, being at times greatly in excess of the littoral movement of sand, forces the channel to the eastward on the line of least resistance. A similar condition exists at the mouth of the Panuco River, in Mexico, where the enormous alluvial deposit in front and to the south of the mouth of the jetties has caused

a northeasterly flexure of the channel, to meet the conditions of least resistance. Mr. Ripley.

The southwesterly movement of the passes along the Texas coast is also a measure of the resultant littoral movement. Galveston Island has very materially shortened its length at the east end by erosion and increased its length at the west end by accretion, within the period of official surveys. Pass Cavallo has moved to the westward within the experience of the writer, so as to completely engulf an old fort built by the Confederates, and has eroded the shore for considerable distance beyond.

The movement of Aransas Pass is remarkable. The following table, compiled from official records, shows the amount by periods:

MOVEMENT OF ARANSAS PASS TO THE SOUTHWEST FROM 1861 TO 1887.

Dates.	Movement.	Time.	Movement per year.
1861 to 1868	1 300 ft.	7 years.	186 ft.
1868 to 1871	650 "	3 "	217 "
1871 to 1875	400 "	4 "	100 "
1875 to 1878	1 500 "	3 "	500 "
1878 to 1882	700 "	4 "	178 "
1882 to 1885	150 "	3 "	50 "
1885 to 1887	75 "	2 "	37 "
1861 to 1887	4 775 ft.	26 years.	184 ft.

An inspection of this table shows that the southwesterly movement of this pass, although not uniform, has been continuous for 26 years, and the topography of St. Joseph Island indicates that this movement has progressed, from a point near Lydia Ann Island, a distance of $2\frac{1}{2}$ miles from its present location probably within the present century. The extraordinary movement from 1875 to 1878 was probably due to the storm of 1875, which was one of remarkable severity, and the small movement since 1882 has been due to artificial works which, since their completion, have rendered the position of the Pass stable.

The theory advanced by the author as a basis for the design of the curved breakwater partially constructed by the Aransas Pass Harbor Company is entirely new to the writer, and if it were correct the designers of the work would certainly be subjected to the ridicule of the profession. A concise statement of the theory upon which this work was designed is to be found on page 33 of House Document 137, 55th Congress, 2d Session, from which the following quotation is made:

"In plan it will differ from the usual form of jetty or breakwater, being detached from the shore and located on the bar to the 'wind-

Mr. Ripley. ward' of the channel. Its axis will be curved (compound and reverse) to produce reactions similar to those found in the concavities of streams, and having radii sufficient to maintain channels of the requisite depths, as revealed by existing curves and their resulting depths of over 30 ft., now found inside the bar. It is designed to fulfill the fundamental conditions of (*a*) arresting the littoral drift; (*b*) admitting the full tidal prism to the interior lagoons; (*c*) controlling the ebb currents and producing a reaction across the bar; (*d*) changing the condition of equilibrium of flood and ebb currents in favor of the latter; and (*e*) of affording aids to navigation by a structure of only half the length of the usual convergent or parallel jetties in pairs."

The partial execution of this work has fully confirmed the correctness of this theory, both as to results and economy of execution.

If the author has really proved, as he says, by a series of experiments that the littoral current at Pass Cavallo comes from the south, it would be a valuable addition to his paper to include that demonstration in his final remarks.

Mr. Sweitzer. N. B. SWEITZER, Jr., Jun. Am. Soc. C. E.—It was not the intention of the author to enter into the merits of the different systems of jetty construction, nor did he intend to cast any reflections on Mr. Cameron, and his engineers, Messrs. Haupt, Wisner and Ripley, as to the merits of their reaction jetty at Aransas Pass, Texas. That the jetty was well planned and the idea novel, there can be but little doubt, and if it had been placed on the south side of the Pass instead of the north, with the spur jetty north of it, in the opinion of the author, the promised depth would have been obtained. In this connection it should be stated that the last Government survey of the Pass, in the fall of 1888 (of the instrumental work of which the author was in charge), showed no positive evidence of a "jetty crossing the work" and stopping erosion on the crest of the bar.

The author, on behalf of the contractor, and Mr. Pitts for the company, had joint charge of a survey made about four months previous to this, which revealed nothing positive as to this old jetty. There is a supposition, but no positive proof, that this jetty still exists.

Considerable stress is put on the fact that the inlets on this part of the Texas coast are moving toward the south, and, therefore, the littoral current also must necessarily move south.

"It is generally assumed that the motion of a body is in the direction of the resultant of the component forces acting upon it. This being a physical law the direction of movement of the inlets (when unrestricted) should indicate the direction of the forces causing that effect. Hence, when it is found that an inlet is moving to the south, it is difficult to understand how the littoral current (producing that progression) flows from the south."

The littoral current moving north along Mustang Island (Fig. 7) intersects the current either at ebb or flood tide coming through the Pass and normal to it, thus changing the direction of the littoral current

(locally) and preventing it from impinging on the south side of St. Joseph Island. The littoral current does not have, at all times, of itself, enough bottom velocity (as mentioned in the paper) to carry the heavy sands, but, held in suspension by the waves, they are carried across the Pass from Mustang Island, from which they are detached by continuous and heavy breakers caused by southeast winds, and deposited on the south shore of St. Joseph Island. It should be mentioned here that winds from the north, or "northers," do not cause surf, and hence the sands are but little disturbed on the Gulf side, but these winds do drive the waters with great velocity out of the bays, thus helping in the southward movement of the inlets. As northers last but a few days in winter, they are but a small factor. The rip-rap placed on the northern shore of Mustang Island has stopped the erosion of that shore line, but in six months, including the summer of 1888, when there was not one day of north winds, St. Joseph Island moved south 200 ft., narrow-

Mr. Sweitzer.

ERRATUM.

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Page 122, twenty-eighth line: For 1888 read 1897.

summer. Fig. 7.

Mr. Haupt's theory regarding the circulation in the Gulf is very interesting, but when it is remembered that after entering the Gulf the rate of speed is about 3 miles an hour and it would be hard to see how this could be kept up for hundreds of miles without exerting a propelling force, it is probably here that the "vertical movement" mentioned comes in.

Mr. Ripley's wind diagram for the harbor of Galveston, coincides exactly with the author's theory, viz., that the littoral current comes from the east in the vicinity of Galveston, where it meets that from the southeast depending on the intensity and velocity of the wind. It should be understood that this current is controlled by the wind, and, like the Gulf Stream proper, is subject to many fluctuations, but that the main course is that indicated by the author.

Mr. Kastl is respectfully referred to the Annual Report of the Chief of Engineers, U. S. A., September 6th, 1887, which states :

"A map of recent survey shows the channel, instead of running southeast in the direction intended, turned off at a right angle and running northeast across the jetty."

Mr. Sweitzer.

The author was misinformed regarding the jetty at Tampico, but if the south jetty was not built first, it should have been.

Mr. Wisner observes that: "The direction of the resultant littoral current along the Texas coast has been established beyond question by careful observers." The author has a letter from Professor Mendenhall, Chief of the Coast Survey, in which he says: "Nothing positive is known of the currents in the Gulf of Mexico."

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 836.

THE NIAGARA RAILWAY ARCH.

By R. S. BUCK, M. Am. Soc. C. E.

PRESENTED MAY 18TH, 1898.

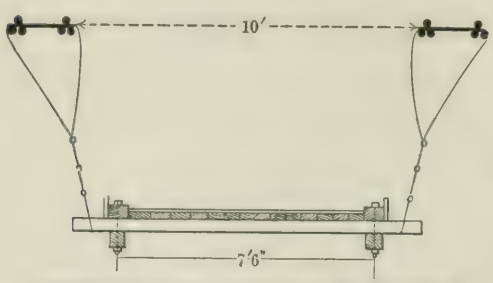
WITH DISCUSSION.

Historical.—Probably there is no bridge site on the Western Continent of greater technical as well as historic interest than that of the Niagara Railway Arch. Each of the former bridges at this site possessed in its day new and striking features, and marked a distinct advance in American engineering.

The plan of spanning the Niagara gorge with a suspension bridge probably first took practical shape when it was suggested to the Hon. William Hamilton Merritt, of St. Catharines, Ontario, by a description of the Freiburg Suspension Bridge in a letter from a friend. This was in 1844. In 1846, through Mr. Merritt's efforts, charters were obtained from the State of New York and the Canadian Government for the construction of the first bridge across the gorge. The scope of the bridge to be built was not then definitely determined, but the charters show an appreciation of the probable development of railroad facilities and the demand for a railroad bridge at this point. At that time there was no railroad to Niagara Falls from the West, al-

though the Great Western, afterward a lessee of the bridge, was in course of construction.

First Suspension Bridge.—In the winter of 1847 the bridge companies made a contract with Charles Ellet to construct a bridge on the site occupied by the present bridge. It was their ultimate purpose to build a railway bridge, but the plan was delayed for some years by the magnitude of the undertaking and lack of funds. Mr. Ellet first threw across the gorge a cable of thirty-six No. 9 wires, on which a light iron carriage was run for about a year and used for the purposes of the subsequent work and for passenger service. From this was developed the earliest bridge (shown in Plate III), which was completed in 1848. This bridge had no stiffening truss. Its towers were of wood, and the expansion rollers consisted of a single wooden cylinder under each group of cables, that is, two cylinders on each tower. A cross-section of this bridge is shown in Fig. 1.



CROSS SECTION OF BRIDGE AT CENTER

Fig. 1.

Mr. Ellet's connection with the work ceased on the completion of this bridge, and he had no hand in planning the railway bridge as finally built in 1853–1855.

Railway Suspension Bridge.—The conception, development and execution of this bridge were the work of the late John A. Roebling, M. Am. Soc. C. E. Both as an engineering feat and as an historical event, Mr. Roebling's great work is of enduring interest. It is fair to say that in the Niagara Railway Suspension Bridge the results of theoretical research were more successfully applied to practical conditions, so far as the strength of materials is concerned, than in any other bridge built, up to that time. A view of this bridge is shown in Plate IV.

Prior to this, Mr. Roebling had built six suspension bridges, but these were for light highway traffic and did not demonstrate his abilities to the extent shown by this work.

The idea of a suspension bridge for railway service met with strong opposition, some of it from high sources. Its opponents insisted that a suspension bridge under the weight of a railway train must neces-

PLATE III.
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sarily be subjected to excessive and dangerous deflection. Mr. Stevenson was at this time evolving the plans for the Victoria Tubular Bridge at Montreal, and opinion was divided as to the comparative merits of the two types. It was no small part of Mr. Roebling's task to overcome the prejudice against his chosen type of bridge. Even after its completion and successful operation for several years, it was still the object of much criticism, most of which was biased and absurd. It is a strange coincidence that these two bridges, the Niagara Railway Suspension Bridge and the Victoria Tubular Bridge, built at about the same time and for the same object, but so totally different in principle, should serve for almost the same length of time and pass out of existence together, to give place to more vigorous successors, better capable of meeting the ever-growing exactions of trade and travel.

It was Mr. Roebling's firm conviction that no other type of bridge was adaptable to the Niagara gorge, and that the suspension bridge was the coming type for long-span bridges. In the first view he was mistaken, for of the three bridges now spanning the gorge, only one is a suspension bridge, and that is being replaced. In the second, he was likewise wrong, and yet in a measure right. The Niagara Bridge was the only railway suspension bridge ever built. Only one other was commenced, and that was not completed. Still, this type has been accepted by high authority as available for spans of a length beyond the reach of other types, for railway as well as highway service. Even viewed in the light of increased experience, and among the vastly multiplied works of the engineer, despite flaws developed by long service, and reconstruction rendered necessary by time and abuse, the Niagara Railway Suspension Bridge will always be considered a monument to engineering skill. It was a great leap toward the high plane occupied by bridge construction at the present day.

Reconstruction of Railway Suspension Bridge.—In 1877, examination disclosed that the outside layers of wires in the cables had corroded at the anchorages. The cables were here embedded in concrete. The strain on them due to moving loads had worked them loose from the concrete, and left a small surrounding space open to the admission of water. This resulted in considerable corrosion, especially underneath the cables, from the face of the masonry back to the shoe.

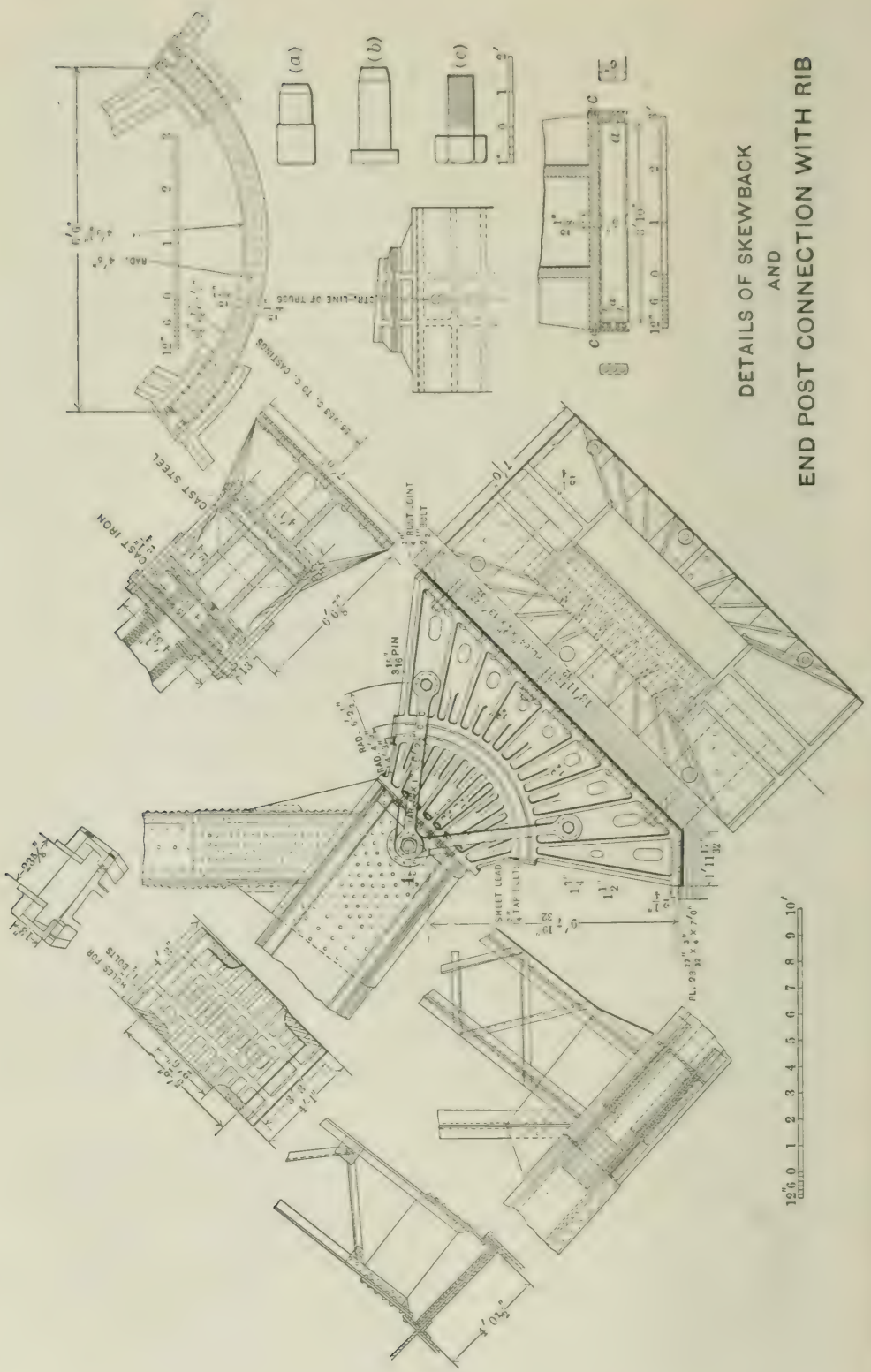
DETAILS OF SKEWBACK
AND
END POST CONNECTION WITH RIB
(a)
(b)
(c)
CAST STEEL
CAST IRON
SHEET LEAD
& TAP FULLY
PL. 23" x 3/4" x 7'0"
12'6" 0 1 2 3 4 5 6 7 8 9 10'

Fig. 2.

PLATE IV.
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BUCK ON NIAGARA RAILWAY ARCH.

The renovation of the cables, and all the subsequent work of renewing the bridge, was designed and executed by L. L. Buck, M. Am. Soc. C. E. The defective wires were cut out and the sound ends connected and spliced under proper stress. The greatest number of wires that required splicing at one end of any of the cables was 65. The wires removed were thoroughly tested to ascertain if there had been any deterioration other than that due to local corrosion. The results indicated none whatever. In fact, some of the wires, corroded partly through, showed a greater ultimate strength per square inch of remaining section than did the unaffected wire.

The wire in the cables of the early Ellet bridge was used in the cables of the railway bridge. The total number of wires in the four cables was 14 560. When the cables were taken down last year the wires were in an excellent state of preservation. In fact, it can be safely stated that, after 42 years of service, they were as sound as when first placed in position. It is interesting to note that when the strands were cut into short lengths they curled up, an indication that they still retained the set given them by the reels on which they had originally been coiled, and that they had not been overstrained.

During the work of repairing the cables it was discovered that parts of the anchorage had been badly strained, by reason of imperfectly formed eye-bar heads, light pins, and imperfect packing. To reinforce these, two new anchorages were put down behind the old ones, on each side of the river, connecting directly with the shoes carrying the strands of the cables. This addition increased the strength of the anchorages about 50 per cent.

While reinforcing the anchorages, Mr. Buck made a careful study of the problem of renewing the stiffening truss. The old wooden truss was very badly decayed and racked, and was fast becoming ineffective. His plans contemplated replacing the wooden truss with a metal truss without interrupting traffic. At the time, this was considered a very daring undertaking, and grave fears were felt as to its safety and success. However, in 1880 the entire plan was carried out without a single serious mishap. This change decreased the dead load on the cables by 178 tons, and permitted a safe increase of live load of from 200 to 350 tons.

A full description of the work of reinforcing the anchorages and renewing the stiffening truss can be found in Mr. Buck's paper on the subject in the *Transactions* of the Society, June 15th, 1881.

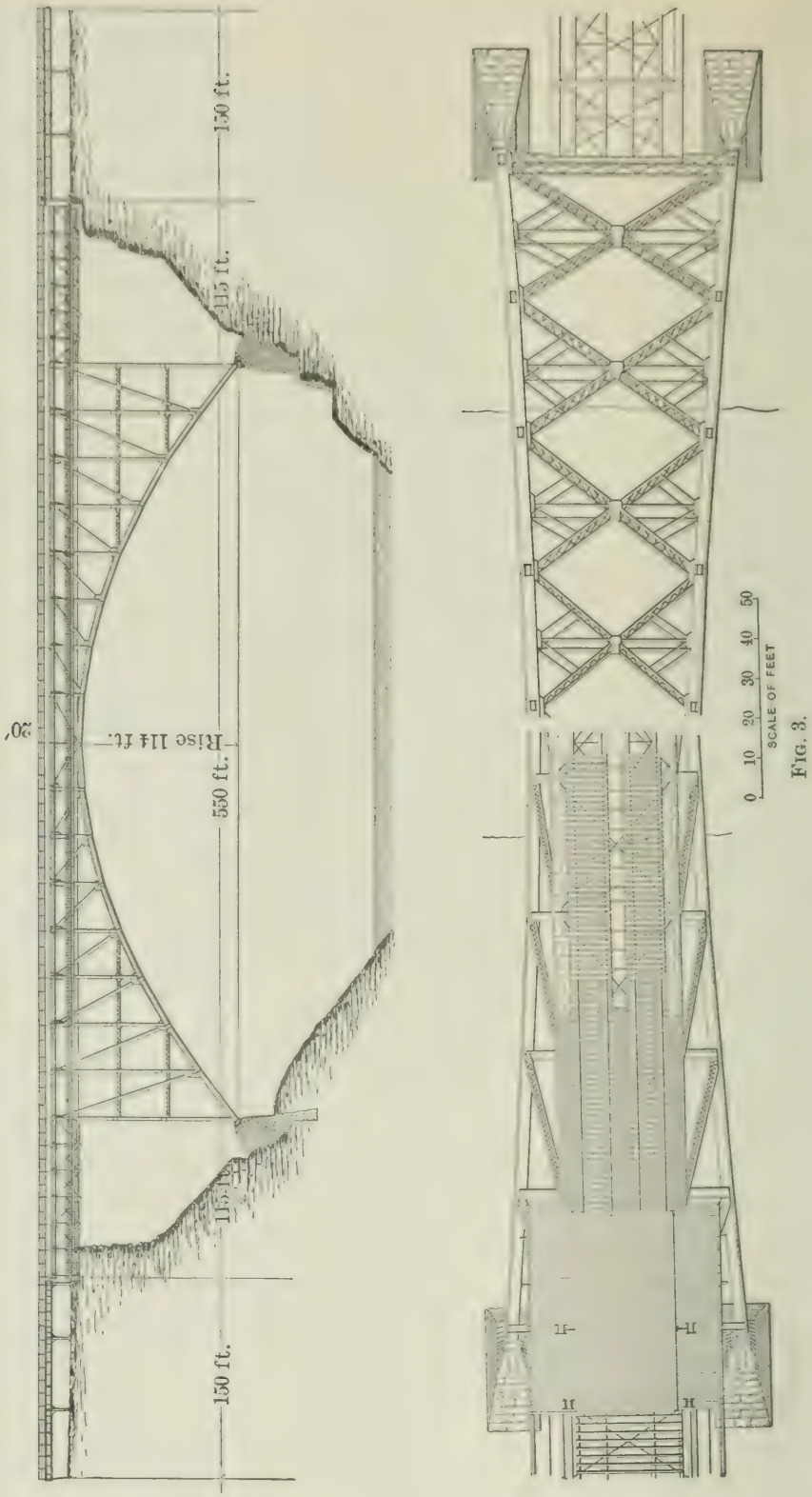
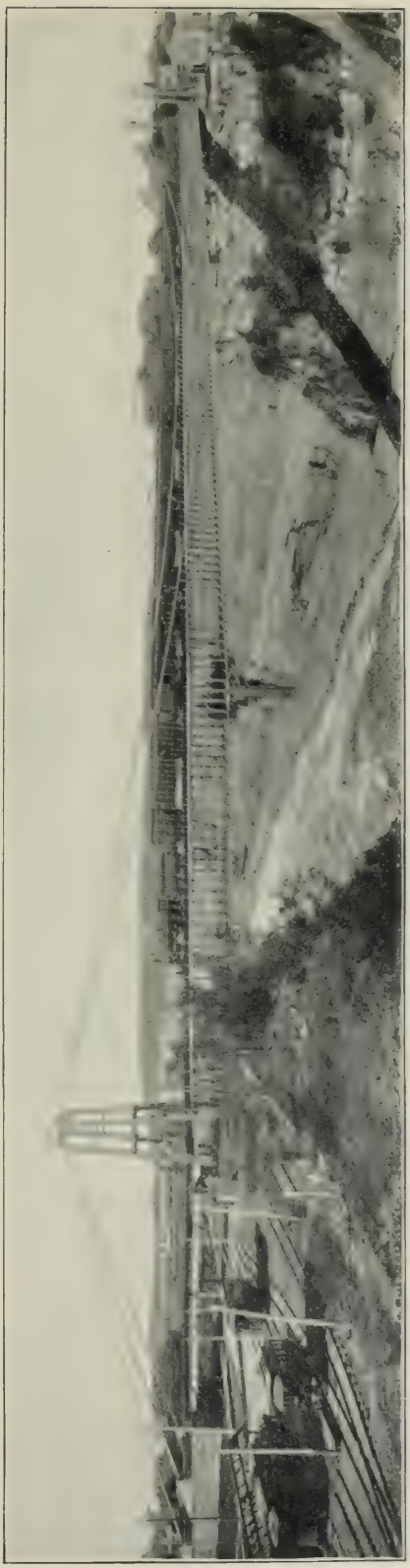

150 ft.
115 ft.
Rise 114 ft.
550 ft.
115 ft.
150 ft.
20'
0 10 20 30 40 50
SCALE OF FEET

FIG. 3.



In 1886 it was decided that safety demanded the renewal of the stone towers carrying the cables. These had for some years shown signs of disintegration, but they were kept in fair condition by replacing defective stones from time to time with sound ones. The disintegration was due to the inferior quality of the stone, and was augmented by the failure of the rollers under the cable saddles to perform their function because of rust. In fact, when the roller beds were taken out, the rollers were found to be fixed immovably in a mass of rust and cement, which had worked its way in from the mortar with which the saddles were originally covered by Mr. Roebling. This caused rocking of the towers under live load and changes of temperature, and greatly accelerated the destructive action of the frost on the masonry. It was at first attempted to preserve the towers by cutting away the defective surface stones and casing them in sound masonry, but it soon developed that the disintegration had penetrated too deeply to be remedied by this means. It was then decided to replace them with towers of iron.

Briefly described, this was accomplished as follows:

The corners of the stone towers were cut away, to admit the piers and legs of the new towers, which were then placed in position and temporarily secured to the former with clamps. The saddles carrying the cables on one tower were then lashed securely to a lifting frame, consisting of bent eye-bars and built beams, and the two cables were raised together by means of six 125-ton hydraulic jacks, resting on the new tower leg. When lifted high enough, the weight of the cables was taken on four short cast columns, one at the top of each tower leg. The old saddle bearings and three courses of masonry were then removed, and the heavy built base to take the bed plates under the saddles was moved into place. The new rollers and bed plates were then placed under the saddles and the weight taken on them. While the cables were being lifted, a period of about eight hours, no trains were allowed on the bridge. This completed the work of reconstruction of the suspension bridge. There then remained nothing of the original structure, except the cables, saddles, suspenders and anchorages. The reconstructed bridge is shown in Plate V.

It was thought, when it was decided to replace the suspension bridge, that the old bridge could be utilized at another site, but when

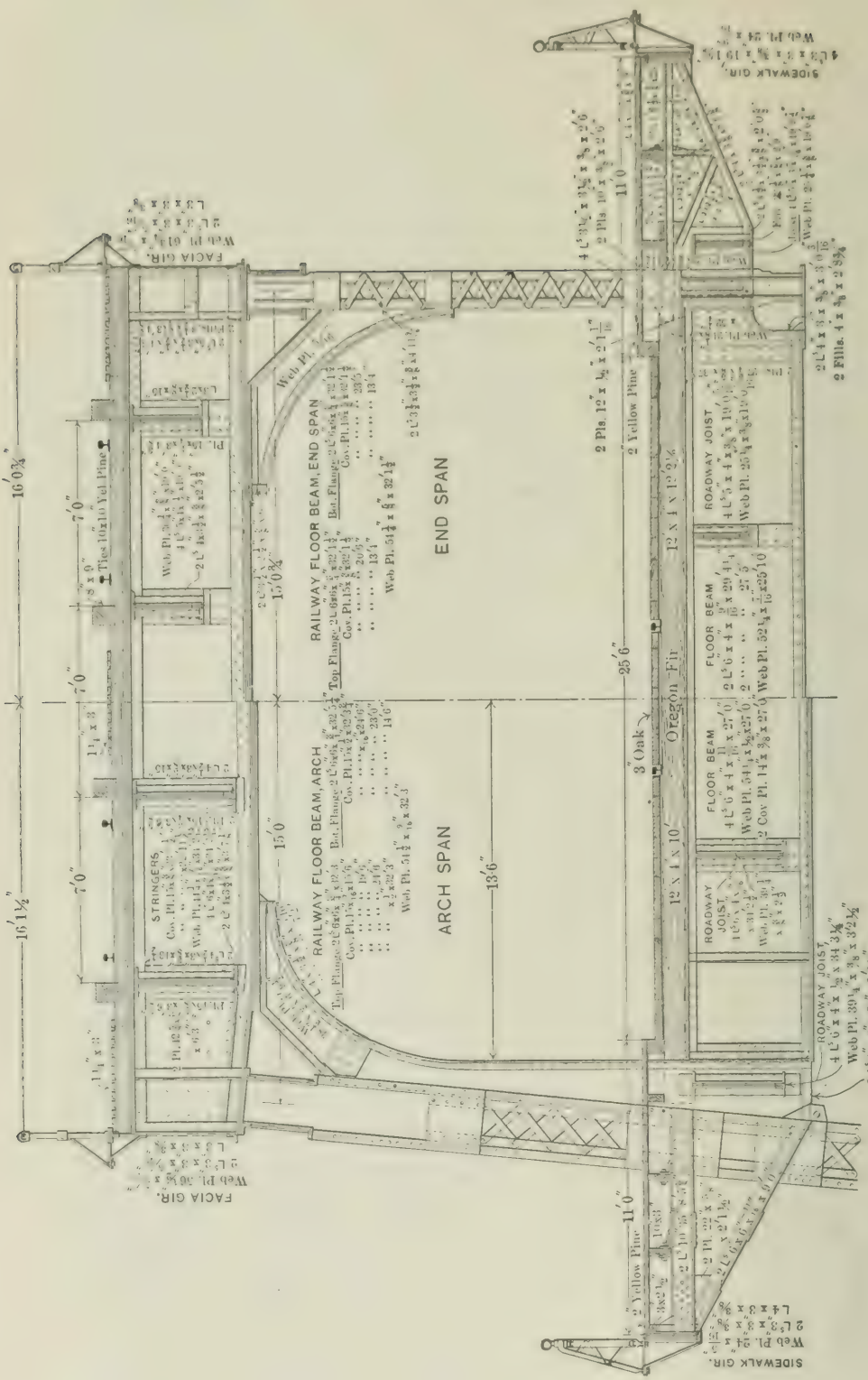

END SPAN
ARCH SPAN
RAILWAY FLOOR BEAM, END SPAN
RAILWAY FLOOR BEAM, ARCH
FLOOR BEAM
ROADWAY JOIST
STRINGERS
FACIA GIR.
SIDEWALK GIR.
2 Pls. 12" x ¼" x 21'-1"
2 Yellow Pine
12" x 4" x 12'-9¼"
3" Oak
Oregon Fir
25'-6"
13'-6"
11'-0"
15'-0"
16'-1½"
16'-0¾"
7'-0"
2 Tiles 10"x10" Yel. Pine

FIG. 4.

PLATE VI.
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the work was done, no site was available; and, owing to the difficulty and expense of taking it down in proper condition for re-erection, the whole structure, except, perhaps, the suspenders and the wind-guys, was consigned to the scrap heap. Some of this material, after going through the furnace and rolls, will appear again in the Niagara Falls and Clifton Bridge.

Arch.—The completed arch is shown in Plate VI. In treating of the present bridge, a simple recital of the facts, as observed by the author, is all that he can contribute towards a discussion of the principles involved in the evolution of a work with which he was fortunate in being associated, during its design and execution. He feels that much of value can be and should be contributed on the subject of steel arch construction from many well-equipped sources, and therefore hopes that whatever is lacking in the paper will be forthcoming in its discussion.

The steel arch has, within the past few years, grown greatly in interest and importance, and is entitled to full consideration. It is rigid, and, at such a site as the Niagara Gorge, is economical beyond any other type. It also stands far ahead of all others, except, that in point of beauty, perhaps its anti-type, the suspension bridge, takes first place in the minds of some.

There is lacking the simple practical treatment of metal arches which has been given to other types of trusses, a treatment which would supply the wants of the engineer who seeks results and cannot afford to master the numerous partial and abstract treatises in order to reach them.

The fund of information on this subject is not scant, but it needs concentration.

The author has derived much assistance from the work on arches,* by Charles E. Greene, M. Am. Soc. C. E., but this does not supply the entire need.

Division of Types.—Arches are usually divided into three general types: 1st, three-hinged; 2d, two-hinged; 3d, hingeless or elastic. Without changing the form or arrangement, or in fact anything other than inconspicuous details, the same general design can be put in any of the three classes, and in each instance will be subjected to radically different stresses and deflections under load. The problem as to which

* "Trusses and Arches Analyzed and Discussed by Graphical Methods."

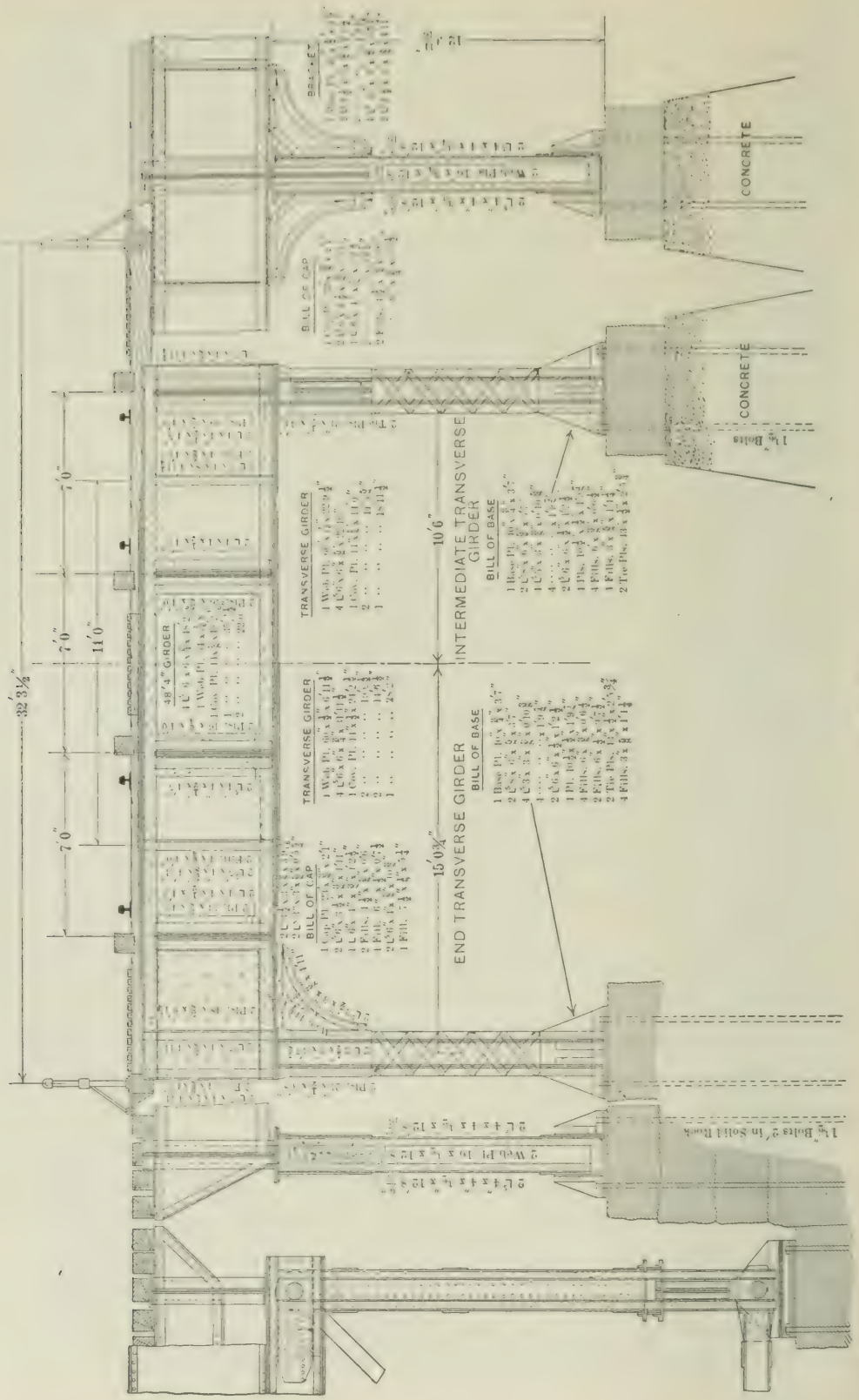

CONCRETE
CONCRETE
1¼" Bolts
BILL OF CAP
TRANSVERSE GIRDER
INTERMEDIATE TRANSVERSE GIRDER
BILL OF BASE
END TRANSVERSE GIRDER
BILL OF BASE
48'4" GIRDER
10'6"
15'03/4"
32'3½"
7'0"
7'0"
11'0"
7'0"
1¼" Bolts 2' in Solid Rock

FIG. 5.

FIG. 1.

FIG. 2.

of the three types is best suited to given conditions is difficult to solve.

The three-hinged arch has been, and will perhaps continue to be, a popular type, on account of its simplicity of computation and adjustment, and the practical absence of temperature stresses. However, what is gained in these respects is at the cost of rigidity, a matter of smaller importance in roof trusses, but of great importance in bridges, especially in those for heavy service. Every hinge is intended to provide for movement, and facilitates distortion under eccentric loading.

A marked advantage in removing the center hinge is that reversal in the web members is greatly reduced, and the top chord is made to carry a larger proportion of the stresses which are otherwise carried almost entirely by the rib.

Hence the question arises: which is preferable, ease of calculation and adjustment, inconsiderable temperature stresses and greater vibration, or greater rigidity with increased temperature stresses and difficulty of adjustment? On similar grounds, comparison can be made between the two-hinged and the hingeless arches.

Niagara Railway Arch.—In the design of the Niagara Railway Arch, the problems presented by the excessive loading to be provided for, by the length of the span, and in the erection, which had to be accomplished without interruption to traffic, all required careful treatment.

The Chief Engineer, after a thorough investigation of all available types, fixed on the two-hinged spandrel-braced arch as best meeting all requirements. In 1882–83, when the subject of building a bridge for the Michigan Central Railway, across the Niagara Gorge, was under consideration, he prepared a design and estimate for a spandrel-braced arch for that work, to be erected in the same manner as the Niagara Railway Arch. This design included the center hinge. However, no opportunity was given to present it to the Bridge Company. The present cantilever was the design adopted and built. The Driving Park Avenue bridge at Rochester, also designed by him, was a three-hinged spandrel-braced arch. The cantilever method of erection was likewise contemplated in the Rochester design, but as the use of false-work there was not impossible, and the method of erection was optional with the contractors, they adhered to the latter method.

After a careful consideration of the vibrations of the Rochester bridge, under loads most calculated to produce vibrations, Mr. Buck

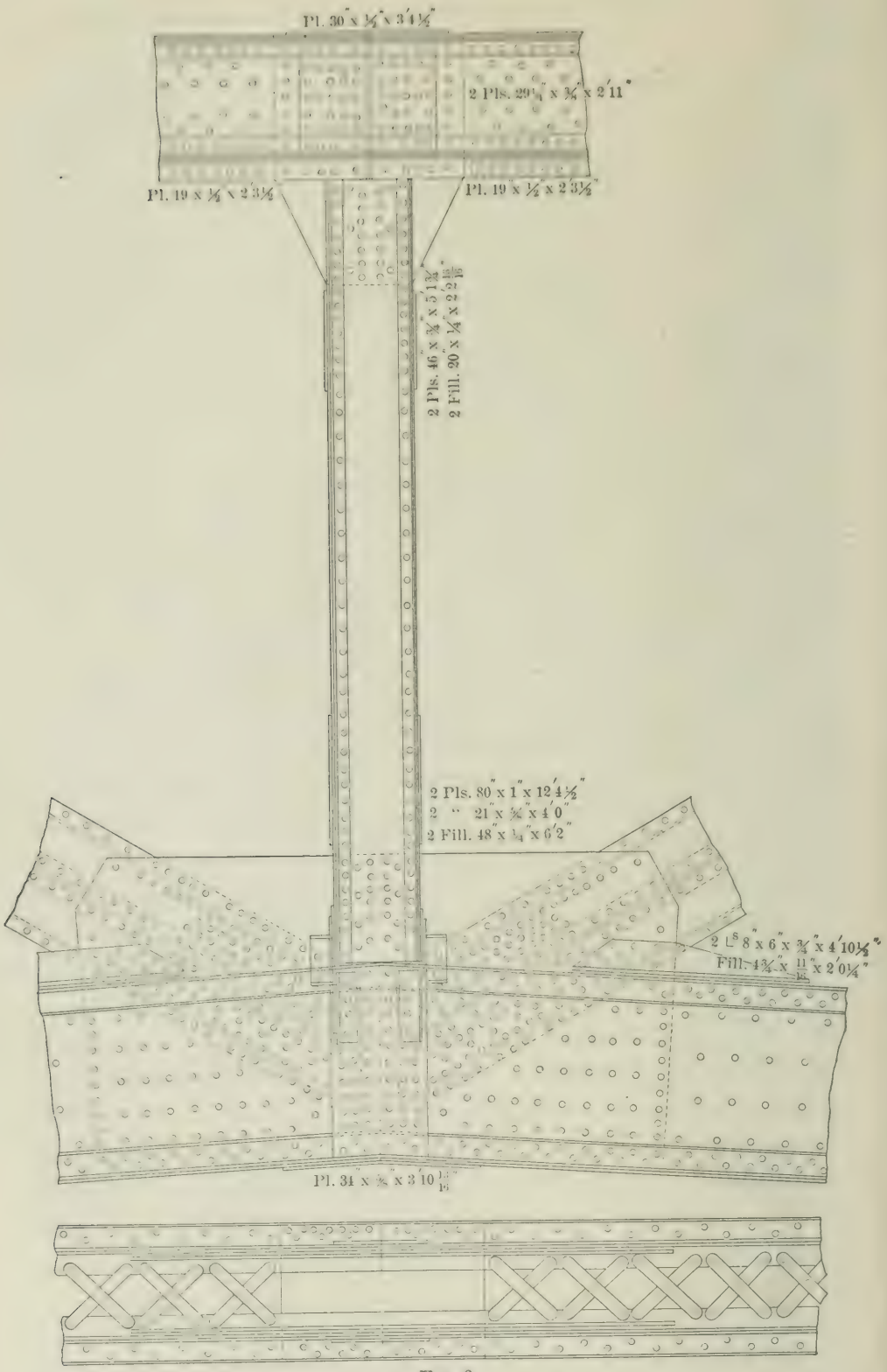
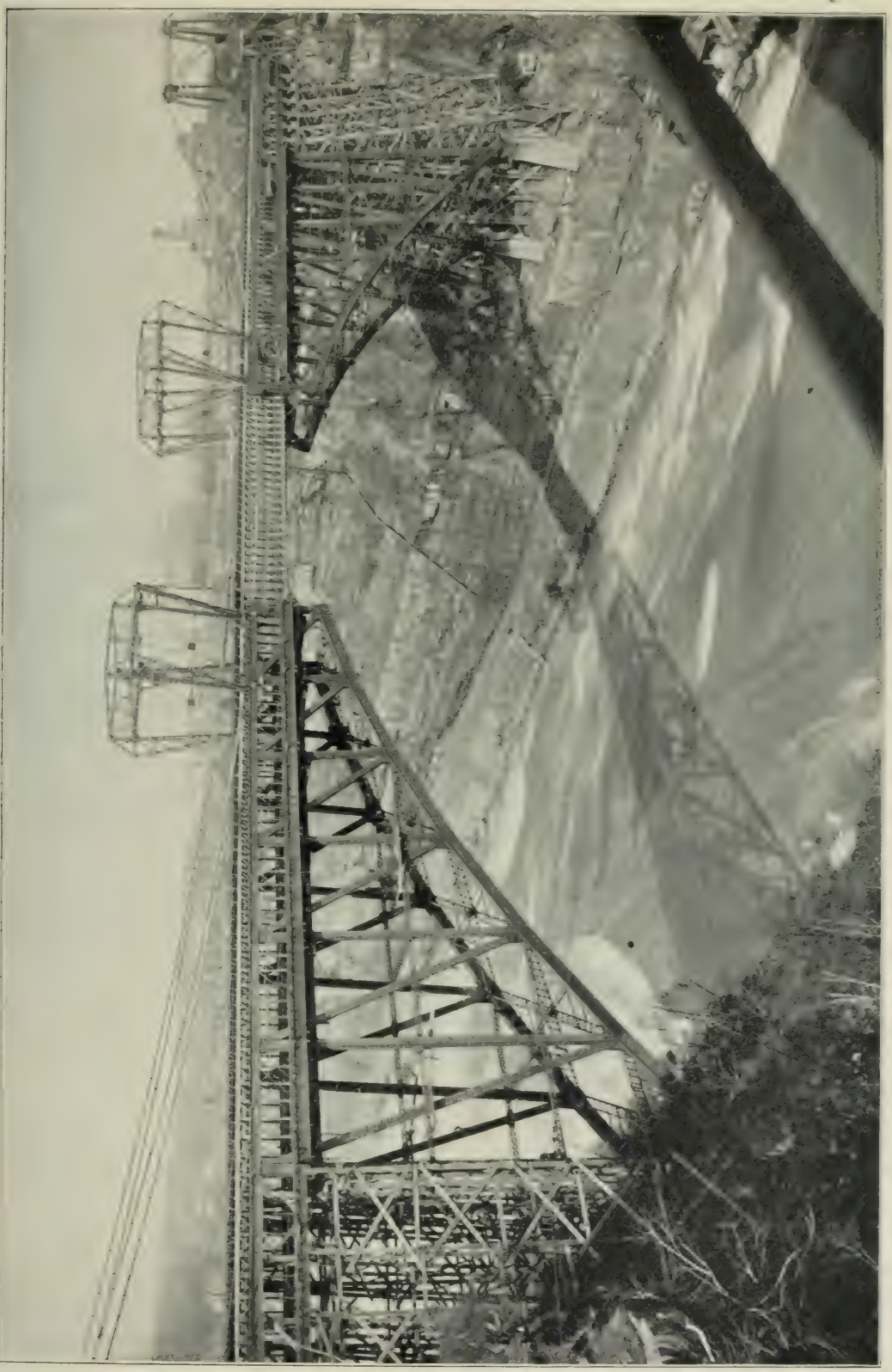

Pl. 30″ x ¼″ x 3′ 4½″
2 Pls. 29¼″ x ¾″ x 2′ 11″
Pl. 19″ x ½″ x 2′ 3½″
Pl. 19″ x ½″ x 2′ 3½″
2 Pls. 46″ x ⅜″ x 5′ 1⅜″
2 Fill. 30″ x ¼″ x 2′ 2 15/16″
2 Pls. 80″ x 1″ x 12′ 4½″
2 " 21″ x ⅜″ x 4′ 0″
2 Fill. 48″ x ¼″ x 6′ 2″
2 L 8″ x 6″ x ¾″ x 4′ 10½″
Fill. 4¾″ x 11/16″ x 2′ 0¼″
Pl. 34″ x ⅜″ x 3′ 10 15/16″

FIG. 6.



decided that it would be best to omit the center hinge in the Niagara Arch. Yet his investigations showed that at Rochester, as probably in most cases, the vibrations seemed, to a person standing on the structure, to be much greater than they actually were. The results in this regard in the Niagara Arch are very gratifying. Vibrations due to trains passing over at a rate of 20 miles per hour are scarcely noticeable, while the generally irresistible jog trot of a horse seldom produces the usual responsive swing. The stiffness thus attained, the author believes, has never been found in any other bridge of equal span. The calculated deflection under a moving load of 10 000 lbs. per running foot is $1\frac{15}{16}$ ins., and the observed deflection under the test load, which was about 6 500 lbs. per running foot, was $\frac{13}{16}$ in.

Method of Calculation.—The absence of the center hinge in the spandrel-braced arch renders the calculation of stresses decidedly more difficult than in the three-hinged type. The method of calculation used was that given in Professor Greene's book on Arches, Chapter XII. However, the sections of the rib in the Niagara Railway Arch are increased so as to be a mean between those required by this method and what would be required if there were a third hinge. This was done to meet any inaccuracy of adjustment due to varying temperature.

Foundations.—The skewbacks of the arch span were located with a view to bring the thrust of the arch on the "Clinton Ledge," a solid stratum of gray limestone, from 12 to 14 ft. thick, about half way between the water and the top of the bluff. Above is a blue shale, and below is the beginning of the Medina sandstone formation, thin layers of shale and sandstone sometimes running into solid sandstone 4 to 5 ft. thick. The bearing comes very fairly on the ledge on the New York side, where the stone was cut at the right angle to receive the masonry directly. But on the Canada side, the bearing was not so favorable and concrete had to be used under the front of the south skewback and under the entire north skewback. The heavy face wall under the New York skewbacks was necessitated by the undue encroachment of the Gorge Road upon the site. The cut made for this road left here an almost vertical face, liable to disintegration on exposure, directly at the front of the skewbacks. The skewback masonry is limestone, with granite copings, entirely of dimension stones with $\frac{1}{2}$-in. joints and strong bond.

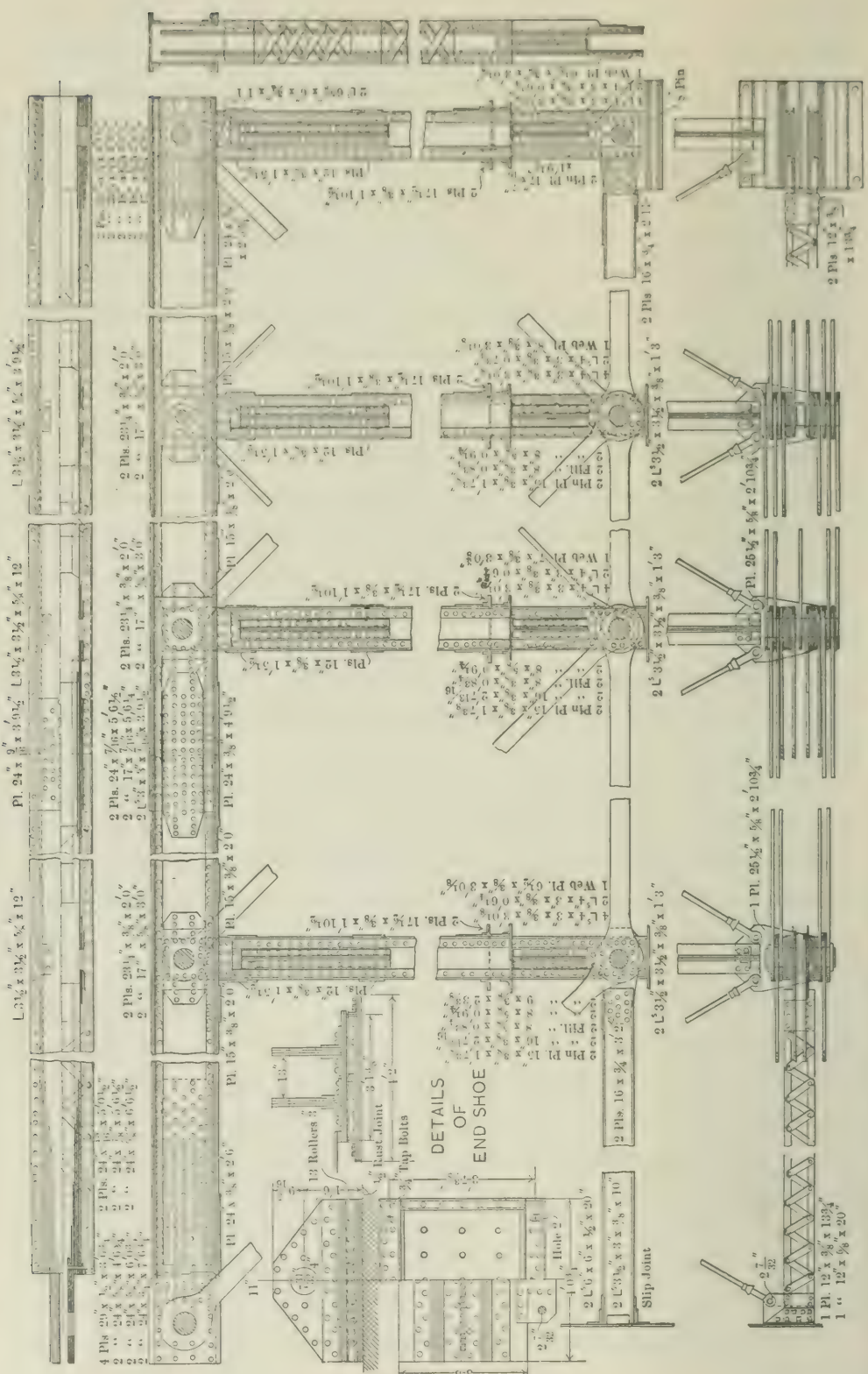

DETAILS
OF
END SHOE
Rust Joint
Pap Bolts
13 Rollers
Slip Joint
Hole 2"

FIG. 7.

FIG. 1.

FIG. 2.

All masonry, except the retaining wall under the New York skewbacks, was put in with rented plant and hired labor, with results very satisfactory, both as to the cost and the quality of the work obtained.

The maximum loads on the masonry are as follows:

On top of coping	339	lbs. per square inch.
Under "	300	" " " "
On concrete	113	" " " "

Rust Joint.—The rust joint, between the masonry and the shoe, is a mixture of 32 parts of cast-iron filings to 1 part of sal ammoniac by weight, very thoroughly rammed. These ingredients and proportions were adopted after experimenting with several formulæ. Thorough ramming is the most important part of the operation.

End Bearings.—The details of the end bearings of the arch span are shown in Fig. 2.

They consist of two steel castings, having between the concave face of the lower and the convex face of the upper a nest of 45 segmental rollers set radially with respect to the center movement at *A*. The axis of the cylindrical bearing faces is likewise at *A*, and perpendicular to the vertical axial plane of the bridge. This form of bearing reduces frictional resistance much as a ball-bearing does, and was adopted to avoid the use of an excessively large pin, with which, movement is rather doubtful of realization.

In placing the rollers, the outside plugs *b* were inserted temporarily, to hold them in their correct radial position and render them fixed. At the top the rollers almost touch each other, and in the wider spaces at the bottom there are $\frac{5}{16}$-in. square bars to cause contact and restrict movement should there be any tendency to overturn. The bars, tap-bolted on both the upper and the lower castings at each end of the roller beds, are further safeguards against undue movement. Thus the rollers act like leaves, and can move either way through only a limited range without binding. As the movement of the rollers, due to moving load and temperature, is scarcely appreciable, there is no danger of the limit being reached. After the first panel of the arch was completed, connected with the anchorage, and swung back to correct position for proceeding with erection, the check plugs were removed and the rollers were thus freed. The center plugs and the guide bars remained permanently.

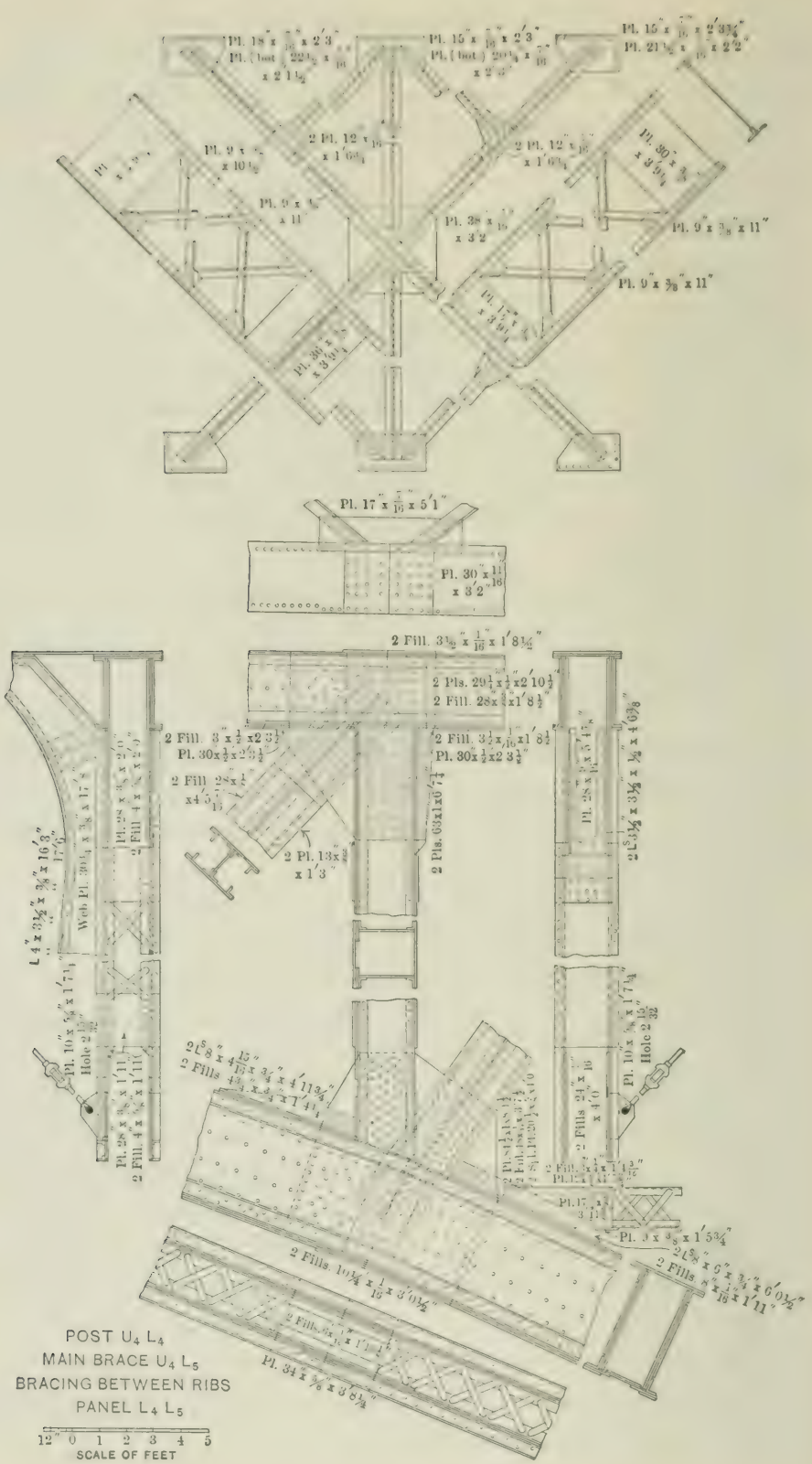

POST U4 L4
MAIN BRACE U4 L5
BRACING BETWEEN RIBS
PANEL L4 L5
SCALE OF FEET

FIG. 8.

The bearing on the rollers, with maximum load on the bridge, is 2 200 lbs. per lineal inch of roller, assuming the pressure to be uniform on all the rollers. The upper and the lower castings were cast each in one piece. The ¾-in. plates on the bottom of the lower castings were intended as a precaution against any possible rupture of the castings.

The manufacture of the bottom castings gave considerable trouble on account of their failure to shrink in the usual manner of steel castings, which was doubtless due to the thinness of the metal and the unyielding nature of the cores.

The eye-bars connecting the rib directly to the lower casting were intended to prevent any possible displacement of the rib or upper casting, a precaution needed probably only during erection.

Trusses.—The trusses, as stated above, are spandrel-braced, with horizontal top chord and parabolic rib. They are battered 1 in 10. The inclination of the planes of the trusses with reference to the end bearings is provided for by a double beveled face on top of the upper casting. This is the only double beveled face in the arch.

The camber of the arch was designed to be 8 ins. at 60° Fahr. It has been observed to range from 10 ins. at summer heat to 7 ins. at zero.

The arrangement and details of the arch span, as well as of the end spans and approaches, are shown in Figs. 3, 4, 5, 6, 7, 8 and 9, and need no further explanation here.

Erection.—The erection was an interesting feature of this work. One of the main objects in view was to maintain traffic, and this was very fully accomplished. Not a single train was delayed, and traffic on the highway floor was suspended only for about two hours each day while the upper floor system was being put in, the time of day selected being that when there were the fewest trains. The lower floor was closed because of the danger to people passing below during the necessarily hurried operations of tearing out the old and putting in the new upper floor.

The deflection of the old bridge under moving load, the constant passing of trains, and the scant clearances at many points were considerations demanding close and constant attention; but as each anticipated difficulty was reached, it usually lost much of its formidable aspect. Besides, there was the comforting assurance that

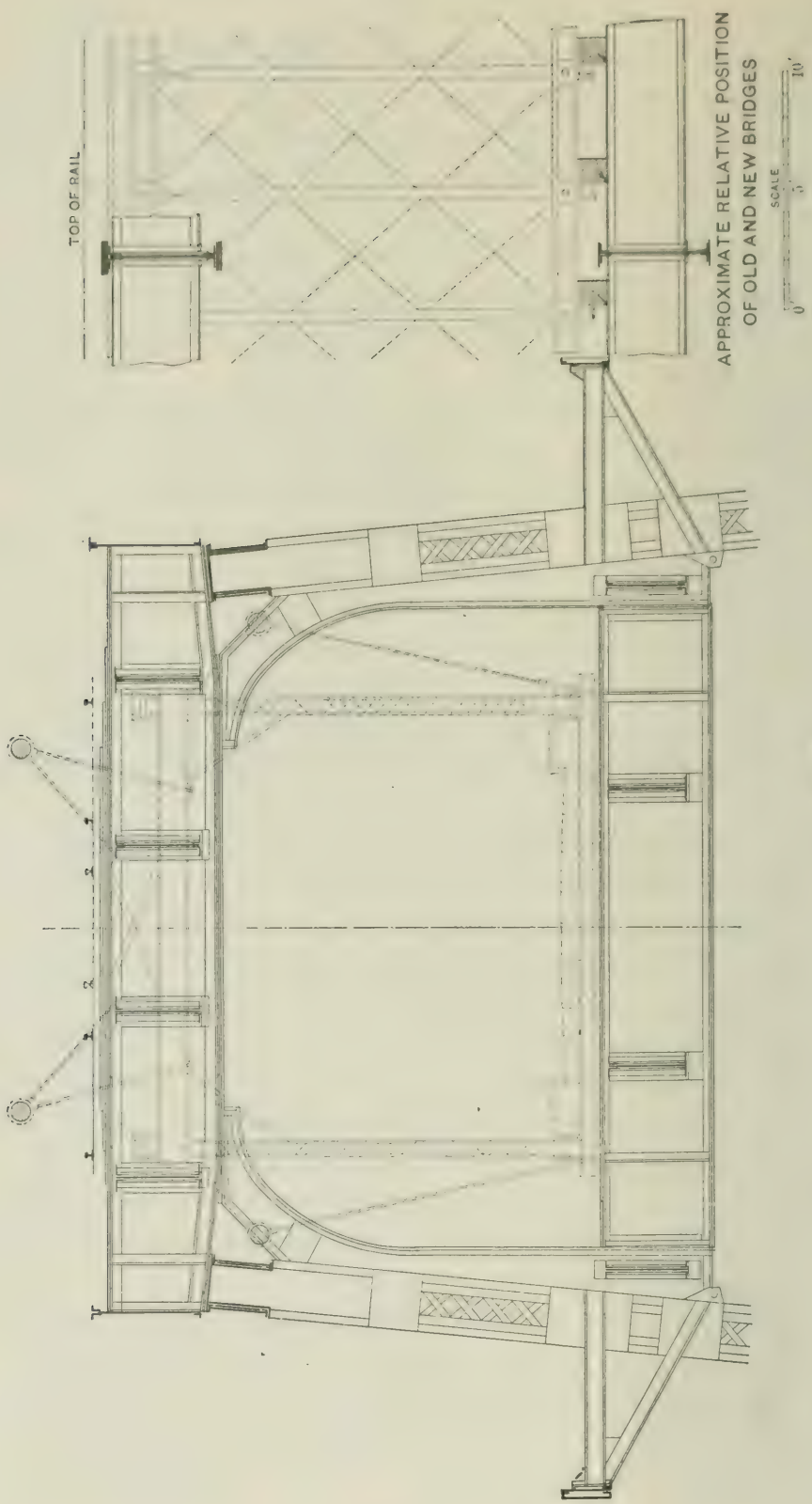

TOP OF RAIL
APPROXIMATE RELATIVE POSITION
OF OLD AND NEW BRIDGES
SCALE
0
5
10'

Fig. 9.

FIG. 1.

FIG. 2.

what had been accomplished with the old bridge under considerably less favorable circumstances ought to be accomplished again.

Briefly, the principle of erection was to build out the two halves of the arch as cantilevers anchored to the solid rock on top of the bluff, by means of adjustable anchor chains connecting with the arch at the top of the end post. The anchor chains consisted of the top chords of the end spans, such of the eye-bars of the end spans as were adaptable to the purpose, and such additional eye-bars and slabs as were necessary to complete the connections. The slabs were used as a matter of economy to serve as short eye-bars. The anchor chain was brought from vertical to horizontal by means of the "spider" shown in Fig. 10, which also shows the principal details of the anchor and the adjusting toggle. This is also shown in Fig. 1, on Plate IX. The anchor pits were cut into the solid rock, back near the anchor walls of the old bridge. They are shafts 3 ft. x 6 ft. in section, and 19½ ft. deep. Chambers were excavated at the bottom of sufficient size to admit the anchors. These anchor pits had to be excavated with great care, to avoid shattering the surrounding rock, and the work was done by hired labor. Border holes were drilled as closely together as possible, to the full depth of the shaft, and the core was then blasted out with light charges of dynamite.

After the anchors and the first two sections of the anchorage chain were placed, the anchor pits were filled with concrete to the top of the rock. Although no provision was made to allow the eye-bars bedded in the concrete to stretch without interference with the concrete, no cracks appeared on its surface until six panels of the arch had been completed. Then some very slight cracks were observed at the corners of the outside bars, but these showed no increase as the work continued to the center.

Adjusting Toggle.—The principle of the adjusting toggle is not new, but its adaptation to this case was very effective. Its operation is apparent from the figure. The right and left screw was turned by hand with capstan bars. Some doubt was felt as to its ability to lift as well as to lower the load coming on it from the weight of the half spans. But this was done without difficulty, nineteen men working each screw.

Erecting Plant.—Under the plan of erection originally contemplated, material was to be conveyed and placed by means of cable-ways sup-

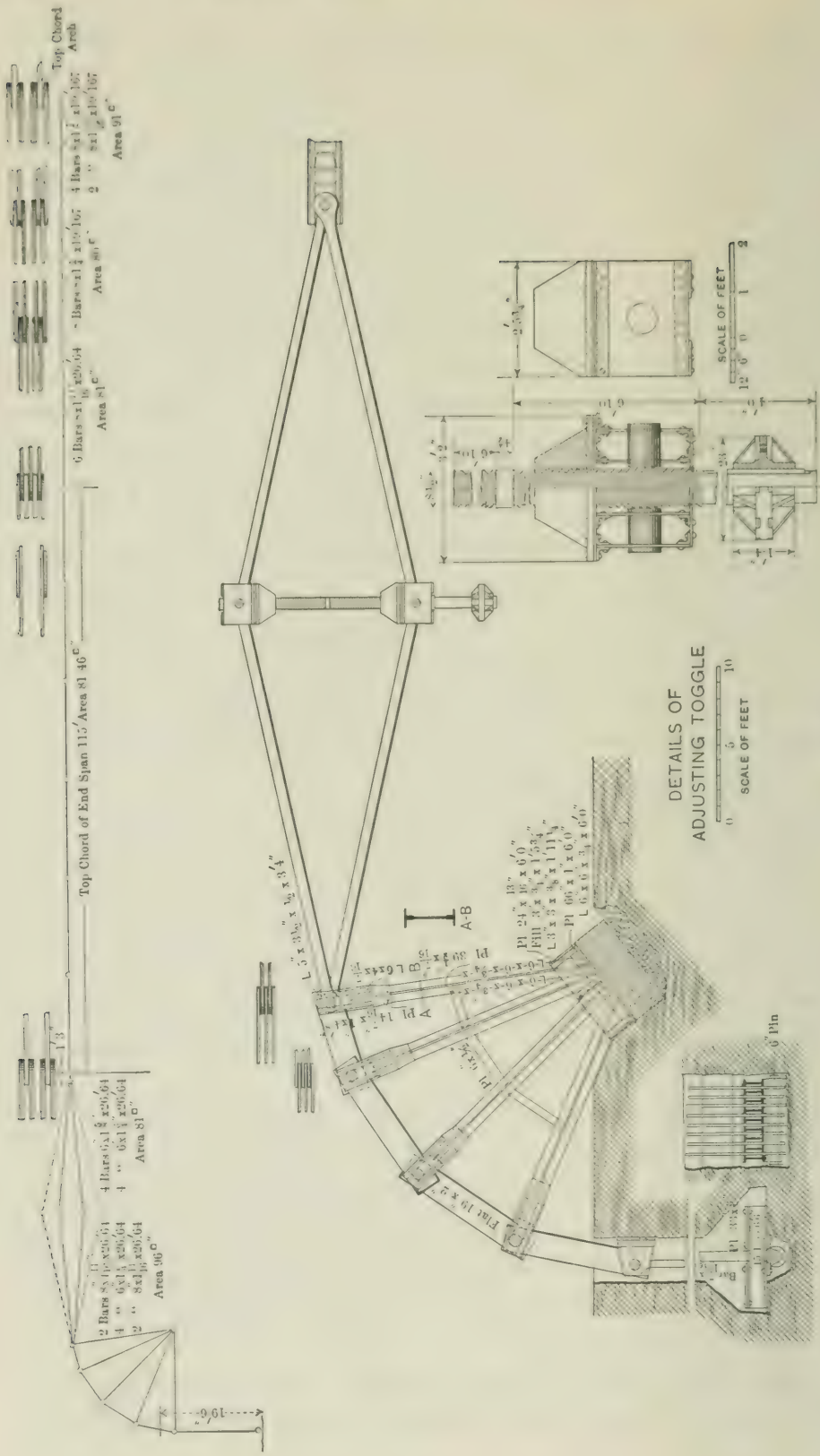

Top Chord Arch
Top Chord of End Span 115' Area 81 46 C"
Area 96 C"
Area 81 C"
Area 91 C"
Area 80 C"
Area 81 C"
SCALE OF FEET
DETAILS OF ADJUSTING TOGGLE
SCALE OF FEET
A-B
6" Pin

FIG. 10.

ported on the towers of the old bridge. This method was used with very satisfactory results by Mr. Buck in the erection of the first Verrugas Viaduct, in Peru, but owing to the large cost of a plant suitable for handling heavy and unwieldy pieces, and their limited experience in using it for such purposes, it was proposed by the contractors to use travelers resting on the top chords of the arch, and the change was sanctioned by the Chief Engineer. The anchorages were strengthened to accommodate the additional weight. This erecting plant proved very safe and efficient. The two sides were entirely independent of each other, furnishing two points of progress, and when there was no outside cause of delay, the erection proceeded rapidly. There were two engines to each traveler, placed in the towers of the old bridge at the level of the railway floor, this being a good point of observation, and well out of the way.

Travelers.—The metal travelers required considerable special treatment, to clear the cables, and furnish the necessary clearance for trains. They are shown on Plates VII, VIII and X. The heaviest piece handled on this work weighed 32 tons, but the capacity of the travelers was considerably greater. For handling the rib members, special clamps were used to make them lie at the angle of the batter.

Progress of Work.—The false-work for the end spans was first erected, and the travelers raised on the outer bents, in which position they handled the skewback castings and first panels of the arch. The first sections of the rib rested on light false-work until the end posts and the braces in the first panel were placed. This much of the first panel was then lifted and held clear of the false-work, by ordinary tackle attached to the tops of the end posts. The false-work is shown in Fig. 1, Plate VII.

When the first top chord sections and the second pair of posts were placed, the pins were driven at the top of the end posts connecting with the anchorages. The end posts were then given the right inclination by means of the adjusting toggle. The traveler was then moved forward on the first panel of the arch, and in this position was ready for the erection of the second panel.

The material was conveyed to the travelers by means of trucks running on tracks on each side of the bridge. These tracks rested on the false-work as far out as the end posts of the arch span, and from

there to the center on the sidewalk brackets. The track stringers for the railway floor were used to carry these temporary tracks, being placed at their proper panels, ready for raising to final position when the railway floor should be put in.

The erection proceeded in this manner to the center. The lower floor system was put in, along with the trusses and lateral bracing. It was dropped below its normal position sufficiently to avoid the possibility of the weight of the old bridge coming on it, when deflected under passing trains, and thereby putting undue stress on the anchorages.

The closure at the middle was anticipated with considerable interest and some anxiety. The absence of the center hinge rendered great accuracy in laying out the work necessary, in order to secure proper closure and distribution of load between the top chord and the rib.

As a safety provision, the center panel top chord sections were not planed to length until six panels of the arch had been completed on each side, and a check measurement had been taken across the intervening space of about 134.5 ft. This measurement was not very satisfactory on account of the difficulties in the way of securing accurate results. The half spans were leaning back from their normal positions, their set and deflection could not be accurately accounted for, and the weather conditions were generally unfavorable. However, it was decided, after taking the measurement, to plane the center chord sections to theoretical length. When the center panels were erected, there remained an opening at the center of 8 ins. due to the two halves of the arch being drawn back, to secure the necessary clearance for placing these panels.

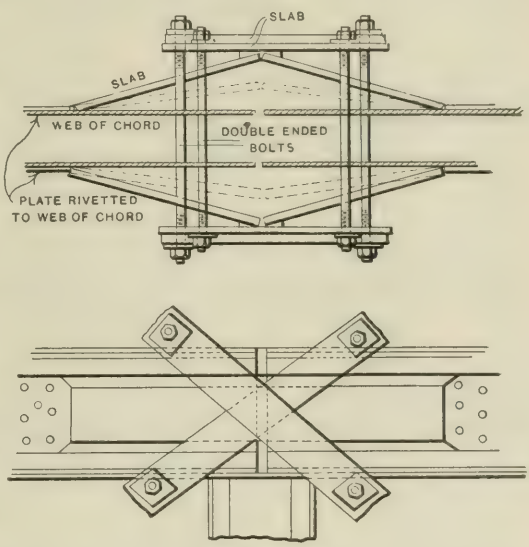
When all was ready, the adjusting toggles were slackened away together. In the proper order of events the top chords should have met first, and then, as those passed from tension to compression, the ribs should have met. But the reverse was the case, the ribs met first, and when the anchorages were entirely slackened off, there was an opening at the center of the top chord of $\frac{1}{2}$ in. This indicated no compression in the top chords at the center, whereas there should have been about 350 tons. The cause or causes of the failure to close, were not, at the time, very obvious, but it was decided that the adjustment could be duly effected after casting off the anchorages. The

FIG. 1.

FIG. 2.

anchorages were cast off and taken apart. None of the joints were riveted up at this time, but almost all holes were filled with drift pins and bolts. Certain of the rib joints were open, the bearing faces being held apart by the drifts and bolts. When these were removed, so as to allow the bearing faces to come together, the opening in the top chord at the center was reduced to $\frac{1}{4}$ in.

It then became necessary, in order to secure the required compression in the top chord, to force it apart at the center and insert a shim. This was done by means of a compression toggle, shown roughly in Fig. 11. This toggle was improvised largely from material on the ground. The chords were forced apart until the opening was 1 in. wide, and a shim conforming to the section of the chord and of this thickness was inserted.



SKETCH OF
COMPRESSION TOGGLE USED
TO FORCE APART TOP CHORD
OF ARCH AT CENTER.

FIG. 11.

Both before and after the top chords were forced apart at the center, levels were carefully taken at each panel point for the purpose of obtaining the exact camber. The results indicated a slight elevation of the camber over the whole span after the adjustment, and in a closer conformity to the theoretical camber.

After the adjustment was effected, and the end spans completed, the lower floor system was raised to its final position. Timbers were laid crosswise on top of the roadway stringers, and when all was ready the stiffening truss was blocked up for its whole length on the new work. This was done between trains. The suspenders were then detached from the cables, and the cables were taken down. The wrapping was cut from the cables with axes, and the strands were cut at the shoes and lowered down, one at a time, on the bridge, where they were cut up for scrap.

After the removal of the cables, the upper floor was put in. In order to do this the upper floor and top chords of the old bridge had

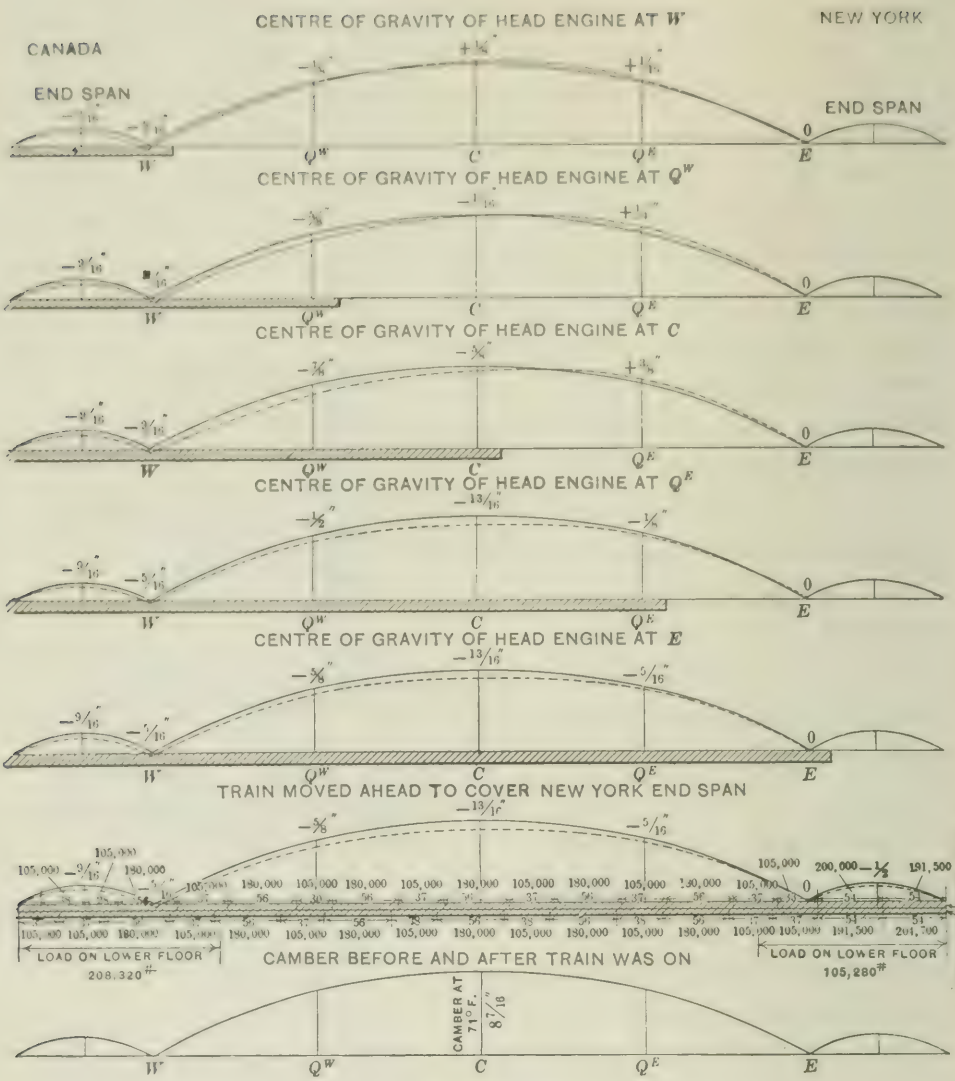


DIAGRAM OF

DEFLECTION OF NIAGARA RAILWAY ARCH

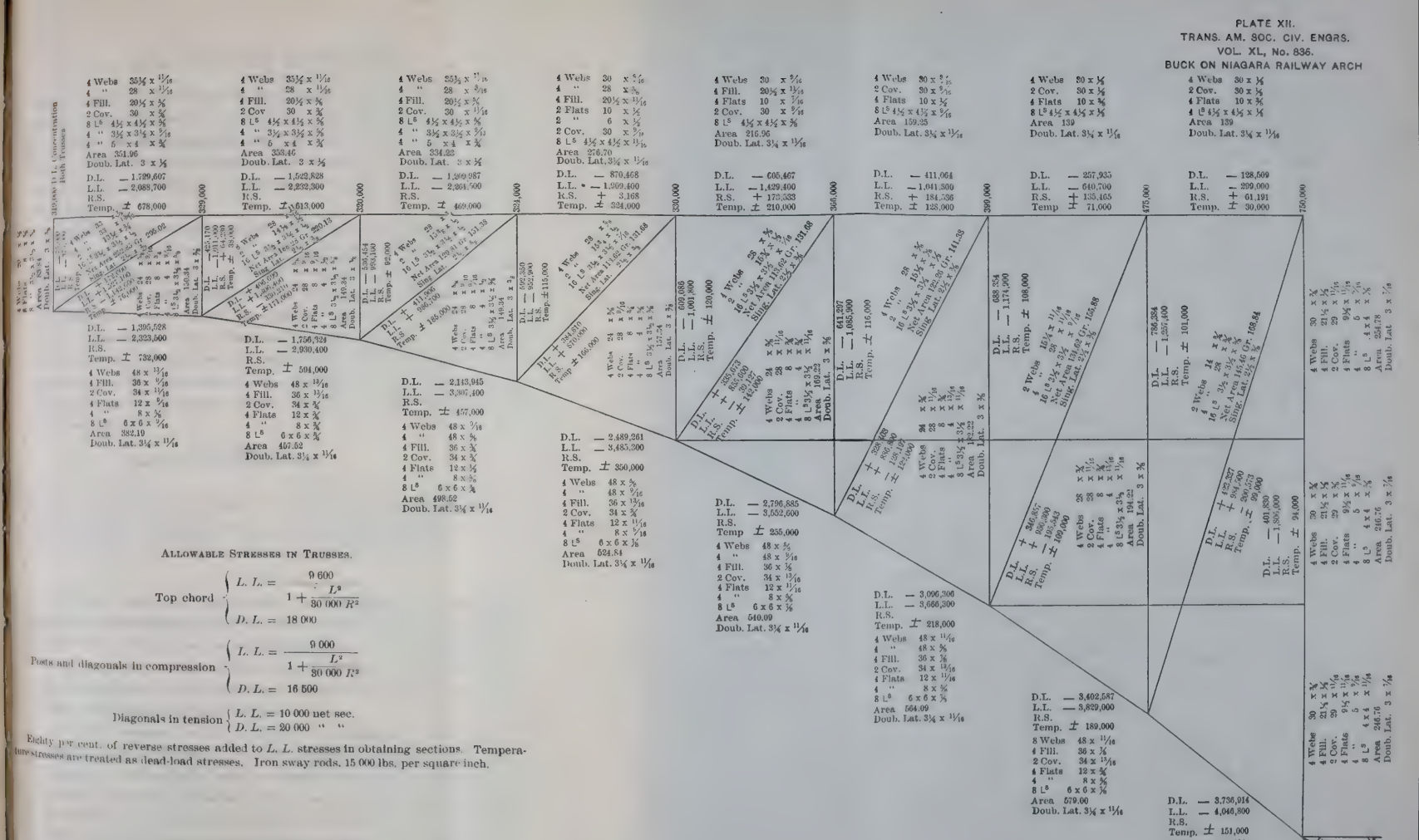
UNDER TEST LOAD OF 2,300 TONS, GR.

JULY 29, 1897.

FULL LINES REPRESENT THE LINE OF CAMBER OF THE UNLOADED STRUCTURE.

DOTTED LINES REPRESENT DEFLECTED LINE OF CAMBER. THE LINES OF CURVE REPRESENT THE MEANS OF THE OBSERVATIONS ON THE TWO TRUSSES. THESE WERE SO CLOSE THAT THE DIFFERENCES WERE NOT WITHIN THE RANGE OF ACCURACY OF OBSERVATION OR PLOTTING.

FIG. 12.



Allowable Stresses in Trusses.

Top chord $\begin{cases} L.\ L. = \dfrac{9\,600}{1 + \dfrac{L^2}{30\,000\ R^2}} \\ D.\ L. = 18\,000 \end{cases}$

Posts and diagonals in compression $\begin{cases} L.\ L. = \dfrac{9\,000}{1 + \dfrac{L^2}{30\,000\ R^2}} \\ D.\ L. = 16\,500 \end{cases}$

Diagonals in tension $\begin{cases} L.\ L. = 10\,000 \text{ net sec.} \\ D.\ L. = 20\,000 \text{ " " } \end{cases}$

Eighty per cent. of reverse stresses added to *L. L.* stresses in obtaining sections. Temperature stresses are treated as dead-load stresses. Iron sway rods, 15 000 lbs. per square inch.

to be removed. In order not to stop traffic this had to be done between trains, two panels at a time. The top chords and track stringers of the stiffening truss were cut into sections, conforming as closely as possible to the panel lengths of the new bridge, and the panels of the new bridge were put in as the sections of the old bridge were taken out.

Operations began at the middle, and after the first day, when only one panel was placed, two panels a day were put in until all were in place. The time allowed for this work was about two hours each day, and the work was always done within the time limit. The same track alignment was preserved, and the same rails and ties were used temporarily after the new floor beams and stringers were in place. When this work was completed as far back as the shore ends of the end spans, the towers were taken down, a high gin pole being used to remove the caps and upper sections, and the traveler to remove the lower sections.

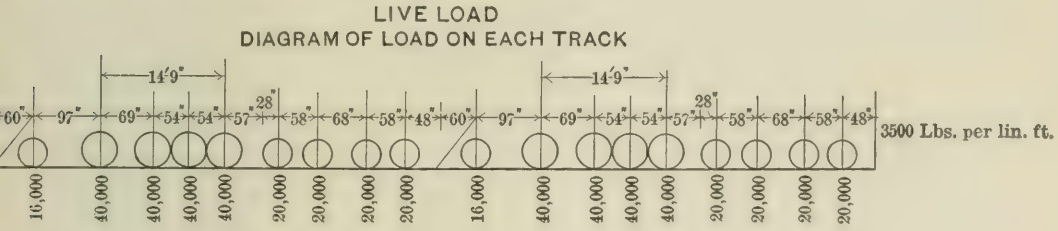


FIG. 13.

After the removal of the towers, the new plate-girder approach spans were substituted for the old ones. The maintenance of traffic here, while shifting, was more difficult than on the main and end spans, on account of the switches to be taken care of; but the change was accomplished without mishap. Plates VII to XI show various views of the bridge during construction.

Test.—On account of the difficulty of securing the full load of 10 000 lbs. per running foot, it was decided to make up two test trains as heavy as were available, and to observe the deflections under this loading. Each train consisted of two heavy Lehigh pushers, four of the heaviest Grand Trunk locomotives at hand and nine coal cars. The cars were of 30 tons capacity, loaded with coal, and had as many rails piled on top as was deemed safe for the cars. The loading is given in detail in Fig. 13. As indicated, some load was put on the lower floor, chiefly on the end spans.

The deflections under the test load are shown in Fig. 12.

The apparent slight irregularities in deflection are probably due more to inaccuracies of observation on account of the humid atmosphere and consequent refraction, than to any real irregularity of settlement of the structure under the load. The arch assumed exactly the same camber after the removal of the load as it had before the load was put on.

Ground was broken for the foundations of the arch span April 9th, 1896, and these were completed September 28th, 1896. The contract for the superstructure was let June 15th, 1896. The work of erection began September 17th, 1896, and the bridge was ready to test and was tested July 29th, 1897. All work on the bridge was completed August 27th, 1897.

In conclusion it should be stated that much credit is due to the Pennsylvania Steel Company, contractor for the superstructure, for care and efficiency in the prosecution of the work.

DISCUSSION.

J. M. KNAP, M. Am. Soc. C. E.—The speaker can urge but one objection to the paper. A good many years ago he had the honor of preparing a thesis on Roebling's Bridge, and predicted that it would stand fifty years, and he thinks that it would have done so had it not become necessary to replace it. Mr. Knap.

J. W. SCHAUB, M. Am. Soc. C. E.—The arch, by the use of curved instead of straight lines, at all times appeals to the eye through its beauty, and so does the Niagara Arch appeal to the eye, provided the eye is fortunate enough to get a proper view of it. The upper arch-bridge now being built is more fortunate in this respect, as, next to the falls themselves, it is now the most striking feature at Niagara Falls. Mr. Schaub.

In erecting an arch on the cantilever principle, without a hinge at the center joint, so many difficulties of vital importance to the integrity of the structure present themselves that the writer is surprised that such methods should be used. In addition to the internal stresses produced in the arch by changes of temperature and any possible yielding of the abutments, there is the uncertainty of closing up the bridge at the center, so as to produce in the arch the same conditions for the dead load as are assumed in the calculations.

If the upper chords of the Niagara Arch had met first, it might have been possible to close up the lower ribs without the use of shims, but this has not been demonstrated. The facts are that the Niagara Arch first closed at the lower joint, and the other joint had to be closed by means of shims; and it is a curious fact that the same thing happened over twenty-five years ago during the erection of the arch-bridge, commonly known as the Eads Bridge, at St. Louis. There, after repeated trials and failures to properly close the upper rib, the lower rib having closed first, the junction was finally made by means of an adjustable member, or practically by the use of a shim, just as was done at Niagara. It should be explained that the St. Louis Arch was a true arch, being composed of two parallel, curved ribs, braced, and was hingeless, with fixed ends. In both cases the dead-load stresses are vitiated to an unknown extent. In the case of the Niagara Arch the stress diagram calls for a dead-load stress at the center top joint of 864 800 lbs. for each truss. The actual dead-load stress now existing in the bridge is practically nothing, compared with the above. It is actually what was put there by screwing up four bolts on a toggle joint, and nothing more. In the St. Louis Arch the case was not so bad, for at the time of closing up the arch the entire superstructure which rests on the parallel ribs was not in place. At St. Louis the lever arm used in screwing up the first closing tube broke, dropping one or two men into the river. The remaining tubes were not screwed

Mr. Schaub. up so tight, and they let it go at that. In attempting to close up the arch one of the members in the bridge was actually ruptured, so that it had to be replaced. This will give some idea of the uncertainty involved in erecting a true arch on the cantilever principle,* In the Niagara Arch the entire dead load was on the bridge, and yet the top joint failed to close. In both cases all this uncertainty could have been avoided by the use of a center hinge, if properly used. The speaker does not recommend a center hinge for an arch similar to that used at St. Louis.

It is argued that a center hinge destroys the stiffness of the arch, and so it does. The difficulty is that the lack of stiffness of the center-hinged arch is not so much due to the hinge as to the fact that the lower chords, or ribs, are not straight. The greater the versed sine of this curvature, the greater will be the moment producing the deformation of the unloaded arm; or, in other words, if only one arm is loaded the other arm will have an amplitude of vibration directly proportional to the curvature of the lower rib. Hence, it follows, that if this curvature is made zero the amplitude of vibration in the unloaded

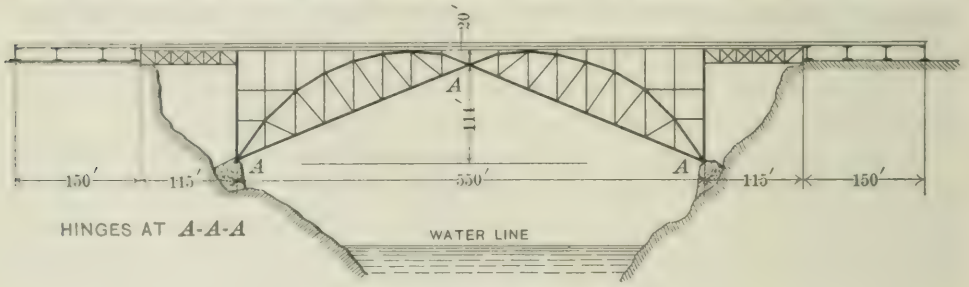


FIG. 14.

arm will be zero. This means that if the lower rib is made a straight line between hinges the deformation of the unloaded arm will be a minimum, and no reversal of stresses will take place.

It is even possible to design an arch so that no reversal of stresses will take place in either upper or lower ribs; then the only reversal of stresses would occur in the web members, and here they cannot be eliminated unless adjustable counter-rods are used. This would give the ideal arch, as far as stiffness is concerned, and, at the same time, eliminate all uncertainty due to the yielding of the abutments and to temperature. Above all things, the stresses would be clearly defined by statical methods, and, after the arch was closed at the center, there would be no question as to the final dead-load stresses.

The speaker begs to submit herewith a sketch, Fig. 14, showing an arch with a hinge at the center, and with the lower ribs composed of straight lines. This form of arch the speaker has had occasion to compute, and finds it a much more economical design than the arch

* "St. Louis Bridge," by Prof. C. M. Woodward, p. 190.

with the lower rib curved. This follows from inspection. The loads travel to their corresponding reactions by the shortest routes, and, by the theory of least work, this must give the stiffest and, at the same time, the most economical structure. Mr. Schaub.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The author states that when the cables of the old suspension bridge were taken down the wires curled up, showing that they had not lost the original set received from the wire drum. The same fact was observed by the speaker when he rebuilt the old Roebling suspension bridge over the Monongahela River, at Pittsburg, Pa., which fact he recorded at the time. * In the Monongahela Bridge the wires had been strained almost daily to half their breaking strength. It would be interesting to learn from the author what the greatest strain had been in the wires of the Niagara Bridge, and how frequently it occurred. Mr. Lindenthal.

That the natural rock abutments of the Niagara Cañon make the arch the proper type for bridging it was long self-evident.

The pains taken in this case to make the bridge very rigid deserve special notice, since this most desirable quality, great rigidity, is too often neglected from notions of false economy. In this respect a comparison with the widely-known cantilever structure of about the same span, and only a few hundred feet above the arch bridge, will not be amiss. The test load for the arch bridge is given at about 6 500 lbs. per lineal foot of bridge, while that for the cantilever bridge was about 4 500 lbs. † Yet the cantilever bridge deflected $7\frac{3}{8}$ ins., as against $\frac{13}{16}$ in. in the arch bridge. On this basis the arch bridge is theoretically thirteen times more rigid than the cantilever bridge. The meaning of this is that the arch bridge will be much more durable.

Several features contribute to its great vertical and lateral rigidity. First of all is the inclination of the arches from the vertical, which greatly lessens the disagreeable lateral swaying, so noticeable in bridges in which the arches are not inclined. Thus, nearest in length of span, and likewise of high rise, is the Washington Bridge, in New York, having six solid-webbed ribs and two spans of 510 ft. each. It is, next to the Niagara Bridge, the heaviest arch bridge in existence; proportioned, namely, for 8 000 lbs. live load per lineal foot as against 10 000 in the Niagara Bridge. Although the Washington Bridge is much wider (80 ft.) than the Niagara (about 55 ft.), yet the former is subject to such considerable lateral vibration as to have caused public comment on its supposed weakness, for which, of course, there is no foundation.

Another instance is the Margarethen Bridge in Budapest, which is a heavy structure of several spans, carrying a wide avenue over the

* *Transactions*, Am. Soc. C. E., Vol. xii, p. 353.

† *Transactions*, Am. Soc. C. E., Vol. xiv, pp. 534–542.

Mr. Lindenthal. Danube. Like the Niagara Bridge, it is of the spandrel-braced type, without the center hinge, and has a buckle-plate floor and a stone pavement. The vibration is very marked, although not so perceptible as in another large arch bridge—the well-known St. Louis Bridge. All iron or steel arch bridges, having the arches in vertical planes, show the same peculiarity of lateral vibration.

On the other hand, arches with inclined planes, such as the Douro Bridge in Portugal, the fine bridge over the Adda, at Paderno, in Italy; the Gruenenthal Bridge over the Baltic Canal, and others like them, show most remarkable lateral stiffness. The bridges referred to have no middle hinge.

That arches without a middle hinge are stiffer than those having one requires no argument. It is true that the calculation of the strains in the arch without the center hinge is laborious, particularly so for the spandrel-braced type; but the labor is well spent for the advantage of a more rigid structure.

The use of riveted connections in the Niagara Bridge is a remarkable deviation from American practice; but that it was a proper choice cannot be questioned, and it is the more creditable to the engineer as the temptation to use pin connections, for greater ease of erection, was one not easily ignored. Increasing experience shows that pin connections should be used in bridges only for members subject to single stresses, either in tension or compression. Since in this bridge almost all members are subject to reversal of stress, riveted connections were the best.

The roller bearings under the arch footings is the one feature which the speaker could not approve; it will hardly find imitators. The author states that the roller bearings under the cables of the old suspension bridge were corroded and had become inoperative. The same thing will happen with the roller bearings of the arches which, however, will do no harm in this case, as the rotation of the arch footings from any cause is nearly *nil*. The speaker would have preferred pin bearings as a simpler and better construction.

The author would add greatly to the value of his description of the work if he would give, in the closing discussion, a synopsis of the calculations and the assumptions on which they were based. Among these are the moduli of elasticity; the panel loads for dead load; whether the top chord at the middle temperature is assumed to be without compression in the middle panels; and whether the calculated deflection (of 1 to $1\frac{5}{16}$ ins. for 10 000 lbs. live load per lineal foot) was obtained by taking account also of the changes of length in the web members. This would be particularly instructive, because the observed deflection ($\frac{13}{16}$ in. for 6 500 lbs. per lineal foot), compared with the calculated deflection, would then furnish a valuable index as to other results of calculation.

In a case like the present, widely different cross-sections for the same design and for the same loads, may be obtained by different computers, unless they are agreed on the premises from which the computations are started. Mr. Lindenthal.

The speaker confesses to be somewhat puzzled by the statement that in closing the arches during erection there should have been a pressure of 350 tons in each top chord. The author does not state that the arches were closed in the winter, or that the temperature was exceptionally low, although the views illustrating the paper show snow and ice on the ground. Even then the necessity for a pressure of 350 tons in the top chord is not clear, without a fuller explanation.

M. Lewinson, M. Am. Soc. C. E.—It would seem from the author's hinge design that as the reaction cannot be assumed to pass through one point, the result will be a rocking arch rather than a hinged arch. In the calculation of an arch like that at Niagara the first condition is that all reaction must pass through one point; that is, through the intersection of the lower chord with the vertical (or sometimes inclined) member. The line found by connecting the common center of gravity of the upper and lower chord sections, taken at various laminæ of the arch, is the line by which the arch should be calculated. The author does not state how he calculated it; therefore, assuming that the reaction passes through the pin, as designed, uneven stresses on the rockers result. It is a question whether the rockers are strong enough to withstand the strain. That the author has not been confident of his calculation of this problem is shown by the fact that he also calculated the arch as a three-hinged arch, and made corrections accordingly in the upper-chord stresses, which is not at all warranted. In a three-hinged arch, if it is advisable to put more compression in the upper chord, it is necessary only to locate the hinge in this chord. Assuming *a priori* all the points as correct, there is no necessity for trying to fill up the gaps in the theory of an arch which is entirely unlike an elastic arch. A three-hinged arch is a static arch, and that under discussion is an elastic arch. The stresses in an elastic arch are determinable, and the addition of material at the inflection points of such an arch is superfluous. Mr. Lewinson.

F. W. Skinner, M. Am. Soc. C. E.—This bridge is very interesting in comparison with the other equally large and important bridges of the present era, as embodying in itself a sort of epitome of the advance of long-span bridge construction from the first instances of it up to the present design for heavy railroad work. It represents four stages: First, there was the simple suspension bridge, without any pretense of stiffening. Next, the John A. Roebling bridge with its stiffening truss, imperfectly designed and more imperfectly constructed. Third, the development of that bridge into a thoroughly stiffened suspension bridge, and last, its entire replacement by a Mr. Skinner.

Mr. Skinner. wholly different type of construction. There is also to be noted the not less radical and striking advance in materials of construction and methods of work. The fact that owing to its location it has been visited by more engineers and others than almost any other bridge on this continent, and that since the first construction of the Roebling bridge the developments have been entirely under the direction of one man who has undertaken some of the most difficult, delicate and hazardous tasks, materially adds to its interest.

In the design of the present bridge the features that seem most striking are the excellent arrangements for the skewbacks, the elimination of the excessively large pins, the extension of the bearing surface at skewback joints and the avoidance of friction, and the portal bracing in the cross-bents in the upper floor-beams. These are most excellent features and are not seen often enough in similar work.

Mr. Lindenthal adverted to the riveted connections, and it may be apposite in that connection to refer to another arch bridge, lately built in Shenley Park, Pittsburg—the Panther Hollow Bridge. This bridge, which is of large span, was built with both pin-connected and riveted joints, was assembled on falsework and the primary connections made with pins. After the bridge was in position, the center panel connected and the arches swung, so that the pins were assumed to carry all the dead-load strains of the bridge, riveted gusset-plate connections were made at all the panel points; the intention being to so proportion and drive them that they should carry only live loads.

The speaker hopes that the author, in closing the discussion, will tell how many men were required to operate the toggle joint for lowering the bridge, how many were required for raising it, and about what speed was obtained; also why it was inadmissible, if it was inadmissible, to use that method of opening the top chord for inserting the key —why it was not used instead of the compression toggle.

It would also be interesting if he could add some data regarding the location of the pier centers; the measurement of the gap in the bridge; if there was anything in particular regarding the making of the core for the casting which he assumes may have had something to do with its shrinking; and whether it was a baked sand core or not.

As the completion of this bridge concludes so interesting a series of constructions, it seems proper to refer to some of the more important features of its predecessors on the site of this structure. It is true that they have have been explained briefly in two papers by Mr. L. L. Buck,* but an extremely brief reference to some of the undertakings involved in the reconstruction will go well with this description of the erection of the new bridge. In the first place, in the re-

* "The Re-enforcement of the Anchorage and Renewal of Suspended Superstructure of Niagara Railroad Suspension Bridge," *Transactions*, Am. Soc. C. E., Vol. x, p. 195. "Replacing the Stone Towers of the Niagara Railway Suspension Bridge with Iron Towers," *Transactions*, Am. Soc. C. E., Vol. xvii, p. 204.

newal of the wires of the anchorage, the ends of the cables, where they unite with the shoes and anchor chains, were exposed and a large number of the outside wires were cut out under strain. Small sections were removed, the ends were scarfed and slightly notched, spliced pieces a little longer than the gap were added, and the wires were put under strain. The strain was measured by deflecting the uncut wires an inch or an inch and a half, and using this as a test for deflecting other wires to get a corresponding strain. The wires were so carefully spliced that they developed almost or quite the maximum strength of the original wires. The old anchorages were found to be somewhat deficient, both in the sections of the members of the chains and in the sizes of the anchor plates, and they were re-enforced by the building of an additional new anchorage for each end of each pair of cables. The new anchorage was built and sets of chains were attached to the shoes of the old anchorages at their connection with the cables, and the anchor chains were adjusted. They were put on by expanding them by heat until they were of a sufficient length, and the strains were measured and adjusted by an ingenious application of the principles of the modulus of elasticity, the bars having first been pulled in machines and the extensions for given strains noted. Both of these works, involving the exposure of the most critical part of the bridge, were carried on without material interruption to traffic and without in any way impairing the integrity of the bridge. Later on, in 1879–80, Mr. Buck proceeded to a still more delicate feat in the renewal of the suspended superstructure. In this case the old suspended superstructure amounted practically to a stiffened tube, a sort of combination truss with wooden compression members and iron rods. The new truss was designed on lines enabling it to practically fit inside of the old one. First the lower floor beams were all renewed. The bridge was then stripped as much as possible of unnecessary weight. Then 150 or 200-ft. lengths of the new lower chord were put in. Fortunately the old bridge members were nearly all double, so that half of them could be cut out without entirely destroying the efficiency of the bridge, and this was done in the floor beams, and later, after the lower chords were placed, in setting up the new vertical posts. Then the upper floor beams were placed, the new top chords were put in position, and half of the diagonal members. The trusses were connected up, and spliced at the ends to the old trusses. The intermediate part of the old trusses was removed, and the process was repeated, the entire old suspended structure being thus removed. After the trusses, the new track stringers were put in. After that the bridge was screwed up to the required camber, which, if the speaker's memory serves him right, was a circular arc of a radius of about 5 000 ft. Then the strains in the six hundred and thirty-six suspenders were all weighed,

Mr. Skinner.

Mr. Skinner. each of them carrying about 3 200 lbs. A hydraulic dynamometer was placed on each one and the strain carefully adjusted to correspond with the varying adjustments of the adjacent ones, and it was found that the suspensions in the suspender rods could be estimated by the touch within the limits of the dynamometer readings (25 lbs.).

The work of the replacement of the new truss was carried on under many disadvantages. It was difficult to place the top floor-beams, on account of the cramped position and the interference with the other work, and particularly difficult to place the lower lateral rods. In order to do it the workmen had to sit on the top flanges of the 8-in. lower floor-beams, working under the floors with very small headroom. There were only two men on the job who were willing to do this work, and one of those being discharged, it was a troublesome matter to get all the lower lateral rods in place.

The conditions of building the bridge were far different from those of the present day. The structure was built in a bridge shop lately converted from an old brewery, but with some special tools, and, under expert mechanical guidance, and by zealous and experienced contractors, it was erected without any steam power whatever. The work was done wholly by hand. Notwithstanding this, there was no hiatus in the work, no discrepancy, no trouble, no particular delay, and not a single serious accident.

The further reconstruction of the bridge is outlined clearly in the paper, but following it from the beginning, through the different changes to the present completed structure, with its extreme rigidity, it is a wonderful piece of work, and is a tribute to the courage, watchfulness, persistence and genius of its author, and a great credit to the able and successful execution of this last enterprise by his resident engineer.

From the first repairs of the anchorage, 20 years ago, no part of the complex and unprecedented work was undertaken until every possible contingency was provided for; remedies were devised in advance, not only for the difficulties that did arise, but that might possibly have arisen through mischance and did not, and well-placed confidence was inspired in all who were associated in the work.

The stiffening trusses of 1880 were the first, or among the very first, large bridge trusses in which riveted structural steel was used, and the character of the work throughout the whole period has been at once conservative and advanced practice.

r. L. L. Buck. L. L. Buck, M. Am. Soc. C. E.—The speaker's work at Niagara began 21 years ago. Four years after he went there, there was talk of another bridge just above the railroad suspension bridge, and it was proposed to put in a cantilever. It always seemed as though that was the place for an arch, and a spandrel-braced arch was the one that seemed best adapted to the place, because it was the most convenient

to erect without scaffolding under the main arch; which would, of course, be impossible in a place with a 20-mile current underneath and from 75 to 140 or 150 ft. depth of water. So the speaker designed an arch very similar to that which has now taken the place of the suspension bridge, but did not get a chance to compete on the design, though some parties had said that no doubt there would be an opportunity to do so, but that they would not build the bridge for some time to come. The "some time to come" proved to be just about a week, as in that time, they had let the contract, and were going right ahead. An arch bridge crossing the Genesee river at Rochester, designed by the speaker, was spandrel-braced and had three hinges, but, very much to the speaker's surprise, when the bridge was empty, with the exception of a double team of large horses, which kept time pretty well at a slow jog-trot, the bridge would teeter near the quarters. Mr. Alden, who did the work, said one day, "We must get rid of that teeter." "How much do you suppose it is, Alden?" "Well," said he, "three-quarters of an inch." He was told that if the whole range of the movement was $\frac{1}{8}$ in. it was as much as the speaker expected. A level and rod were procured, and the rod was laid on the sidewalk so that it had an inclination, and the level set up on the shore so that a sight could be taken at the rod. As it was raised up and down it would, of course, make a considerable movement transversely on the horizontal hair of the level, and in this way would indicate pretty fine readings of the amount of the up-and-down movement from the normal position. The movement was less than $\frac{1}{16}$ in. Mr. L. L. Buck.

In Brock's Monument—a stone tower—during a high wind, there is considerable motion which feels as though it was 3 or 4 ins., though probably it is not nearly as great an amount as that.

After building the three-hinged arch the speaker decided that the center hinge, although theoretically giving less temperature strain, was in many respects a mistake, as the reversals in the diagonals and in the top chords are greater, and the top chords give no assistance in supporting the load, their only office being to stiffen each half of the rib. The speaker's idea in this design was that if the stresses in a two-hinged arch were carefully calculated through all the members when the dead load alone was on, and the shortening of members in compression and the lengthening of members in tension accurately worked out, and the increase or decrease in the length of each as indicated by these calculations made; the top chords being in tension and the lower chords in compression in erection, the top chords would naturally meet first when the bridge came together. They would have done so, except for the fact that the connections being riveted, and the sheets held in position by drift pins and bolts, each gave a little at either end; the braces being in tension pulled slightly out of position, and the total movement amounted to considerable when the middle of the

Mr. L. L. Buck. span was reached, while the posts had just the opposite effect. This, the speaker thought, was the reason the top chords did not come together without a shim; but the shim did no harm at all. It required a little more work, but by turning one nut the toggle was compressed on both sides and with a given pull on the wrench, and a careful estimate of friction, the required pressure could be very closely obtained. The result proved the work, because if the pressure had not been right in the top chord a good many of the very long diagonals would have shown more or less tendency to spring sideways. This has never been shown, at any temperature, and it is safe to say that the work is practically correct, and that, there being no pin connection, every member of those trusses is helping to sustain the load. When the full load is on, the diagonals are in tension and the top and bottom chords in compression.

In making the calculations the lower chords at the middle point were lighter than the speaker liked to have them, and the middle portion of the top chords was very heavy; consequently, he increased the middle portion of the bottom chord by one-half of the difference between the three-hinged and the two-hinged arch, for which he is not sorry.

The speaker wishes to say that in addition to the valuable assistance the author rendered in the calculation of stresses, he has filled the position of resident engineer with great energy, excellent judgment and strong loyalty to his chief, and has contributed greatly to the success of the work.

CORRESPONDENCE.

Mr. Goldmark.

HENRY GOLDMARK, M. Am. Soc. C. E.—There is so much that is interesting and valuable in this paper that the writer cannot help regretting its comparative brevity.

Among other points, a more detailed statement as to the method used in determining the stresses would be of much interest. It is to be hoped that the author may give some further information on this subject in closing the discussion.

While all arches with less than three hinges depend on the theory of elasticity, as well as on statics for their stress determinations, this particular form reduces the difficulty of computation to a minimum. Furthermore, a single system of bracing is used, with riveted connections at all points, so that the changes in the lengths of the separate members are purely elastic, with no play due to pin fittings.

The details of the bearings at the skewbacks are also very skillfully designed to reduce friction as much as possible, so that the usual assumption of frictionless hinges is very nearly justified.

Under these conditions, the only indeterminate quantity involved is the magnitude of the horizontal thrust H at the abutments. When this is known the stresses in all the members can be very easily found by simple statical methods.

In this respect the design compares very favorably with ribs having solid web-plates, and also with all forms of arches in which the bracing is arranged in multiple systems. In all of these the theory of stresses is more uncertain and the arithmetical computations more complicated.

The best method of solving all indeterminate frames is, beyond doubt, that known as the Method of Least Work, or Virtual Velocities. This method is mathematically exact, being in fact merely an extension to framework of the well-known mechanical theory of Virtual Displacements.

The first application of this theory to engineering structures was made by Clerk-Maxwell in 1865,* but it has subsequently been developed almost entirely by Continental writers, and made a useful instrument for solving all the more complex questions involving the stresses and deformations of structures. Reference, however, should be made to a valuable paper by G. F. Swain, M. Am. Soc. C. E., which is almost the only contribution to the Method of Least Work which the author has met with in the English language.†

In calculating a two-hinged spandrel arch, such as that treated of in the paper, the only assumption involved in determining the stresses

* *Philosophical Magazine*, 1865.

† *Journal of the Franklin Institute*, Feb., Mar., April, 1888.

Mr. Goldmark. due to external loading is that of a Constant Modulus of Elasticity. For modern structural steel the error introduced by this assumption ought not to exceed 2 or 3 per cent.

The effect of temperature changes, is, of course, more uncertain, as their variation must necessarily be somewhat arbitrarily assumed. There is, however, no difficulty in adopting an extreme range, which will provide for all possible cases that can occur in practice, and even under this assumption the economy of omitting a third hinge is considerable.

The use of a central hinge at the crown of the arch is commonly assumed as obviating the necessity of taking temperature stresses into account. From the observations he has been able to make of a number of large arches, used either as bridges or roofs, the writer is convinced that this assumption is, in most cases, unwarranted. In every arch he has yet examined, including several designed by himself, the friction in the pin bearings was clearly sufficient to stop all rotation and hence to overthrow the condition of frictionless hinges, on which the entire theory of the three-hinged arch is based. In addition to the friction in the usual pin connections, the necessary floor fastenings in both highway and railroad bridges, and the roof covering in buildings, are usually sufficient to prevent all motion.

As an instance of this, the three-hinged arches in several of the World's Fair Buildings at Chicago may be mentioned, in which the two halves of the arch were quite rigidly connected by iron and wooden purlins and lantern girders. As these were entirely continuous across the central hinge, all rotation was effectually prevented.

Mr. Moncrieff. J. M. MONCRIEFF, M. Am. Soc. C. E.—It would add still further to the usefulness of an already valuable paper if some details could be given of the weights of steel in the various parts forming the main span, such as:

The combined weight per lineal foot of the arch frames (ribs, top-chords and web-bracing).

The weight per lineal foot of span of the lateral and sway-bracing.

The weight per lineal foot of the steel floor systems.

The weight per lineal foot of the remaining permanent load of timber, permanent way, etc., etc.

These details, together with a short statement of the allowed unit stresses, would enable one to make a comparison with other types, as to the economy of the structure in carrying the usual running load of 10 000 lbs. per lineal foot.

The writer notes also that no reference is made to wind pressure, either as to its amount, or influence on the structure.

With regard to the erection, the insertion of the shim in the top chords, by forcing the latter apart by means of the compression toggle shown in Fig. 11, appears to the writer to give no guarantee that the

Mr. Moncrieff.

required compression of 350 tons was either reached or not greatly exceeded, and as the permanent stresses in the various members depend, to some extent, on the stress in the top chords, there is, evidently, some uncertainty as to their actual value in the finished structure, over and above the uncertainties due to the assumptions usually made in the stress calculations for such structures as well as to conditions of erection, temperature, etc. That this uncertainty was present in the mind of the designer of the Niagara Arch is evidenced by the sections of the ribs having been increased to be a mean between those required for a two-hinged arch and a three-hinged arch.

It is to be noted that the required compression of 350 tons forms an appreciable amount in relation to the total stress on the top chords, as given on the stress sheet accompanying the paper, assuming that the 350 tons and the figures on the stress sheet both refer to stress on two arch frames together.

With regard to the adjusting toggles in the anchor chains used for erection, the writer devised a similar means of adjustment in 1896 for the erection of the new Redheugh Bridge across the Tyne (England), now in course of construction from his designs.

This bridge is to consist of two spans of 248 ft., two spans of 168 ft., and a number of short plate-girder approach spans. The larger spans are to rest on three braced steel piers in the river and on two masonry piers, one on either river bank, and the erection, at a clear height averaging 87½ ft. above high-water level, is to be carried out without staging or falsework other than is afforded by the old bridge which is being replaced by the new structure. The old bridge is, however, in such a condition that no part of the new structure is to be allowed to rest on it during erection, and it will simply serve as a platform for conveying the material to the point of erection.

The main trusses are to be of the Petit type, and the heads of the inclined end posts of adjoining spans are to be connected together over each river pier by adjustable toggles. The trusses are to be erected as cantilevers on either side of the piers meeting at the center of each of the 248-ft. spans, and in the case of the 168-ft. spans from the river piers for the full span to the masonry shore piers.

As the erection of the main girders and their toggle adjustments is not yet commenced, it is a satisfaction to know that the toggles at the Niagara Arch bridge were so effective, both in raising and lowering the load.

The roller bearings at the heels of the Niagara Arch are, in their earlier life, no doubt, free from some of the objections accompanying the use of large pins, but unless means are provided for periodical cleaning, and proper attention is paid to this duty, there may be a positive disadvantage in having adopted rollers in preference to the much simpler pin.

Mr. Moncrieff. The writer notes that the material of which the rollers is made is not stated in the paper.

The engineer, and all concerned in the work, are to be congratulated on the successful completion of so handsome a structure under difficult conditions of erection.

Mr. Johnson. J. B. JOHNSON, M. Am. Soc. C. E.—The great amount of labor involved in the computation of a spandrel arch of the kind here given creates a demand for a simpler method. The writer therefore offers the following approximate method, and hopes it may have a fair trial at the hands of those who may be called upon in future to design such structures.

Since the great problem is to find the horizontal components of the reactions for the several joint loads, and since all full-spandrel trussed arches would be very much alike, as to general relative dimensions, the writer has taken the reactions as found for this Niagara arch (as kindly sent to him by the author), and has simply drawn lines through the springing representing their several directions, and extended these to intersections with their respective verticals through the several joints of the arch. The broken (dashed) line joining these intersections, b', c', $\ldots\ldots k'$, may be called the locus of the reaction intersections. If now a parabola be made to fit this true locus as nearly as may be, and its equation found in terms of the constants, span, l, rise of arch, r, and depth of crown, D_c, this may be called the "parabolic intersection locus," and it could be constructed in place, as soon as the general dimensions of the arch were known. The reaction lines could then be drawn to the intersections of the verticals with this curve, and the directions, positions and amounts of these reactions thus at once determined; that is to say, all the peculiar difficulties of the analysis for such a structure would disappear if this intersection locus were known.

In the present example, it can be seen by inspection of Fig. 15 that the error in the horizontal or vertical components due to an erroneous direction of the reactions, if these were drawn to the parabolic locus instead of to the true one, would be very small. Taking joint E as the worst case, the horizontal component would be in error about 2%, the vertical component about $2\frac{1}{2}$% and the resultant reaction by less than 2 per cent. The average single error for all the joints would be less than 1% and these would be compensating, as they are of opposite signs. The resulting effect upon the dimensions of the members, for any combination of loads, would therefore be practically zero. For this one case, therefore, the parabolic locus would have served as well as the true locus, and another curve could be found which would fit the true locus still closer if it were thought necessary.

As to using the equation of this parabolic intersection locus for another bridge, as in a new design, there is only this to be said: The

intersection locus for a two-hinged arch of constant depth (parallel ribs) is not very different from the parabolic locus here found. The equation of this latter, when referred to an origin at the center of the line joining the two hinges, or springing points, is Mr. Johnson.

$$y = \frac{32\, l^2\, r}{25\, l^2 - 20\, x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

y = ordinate to locus curve,
l = span of arch,
r = rise of arch,
x = distance out to right or left from the center.

The curve marked a'', b'', c'' ..... k'' is this locus.

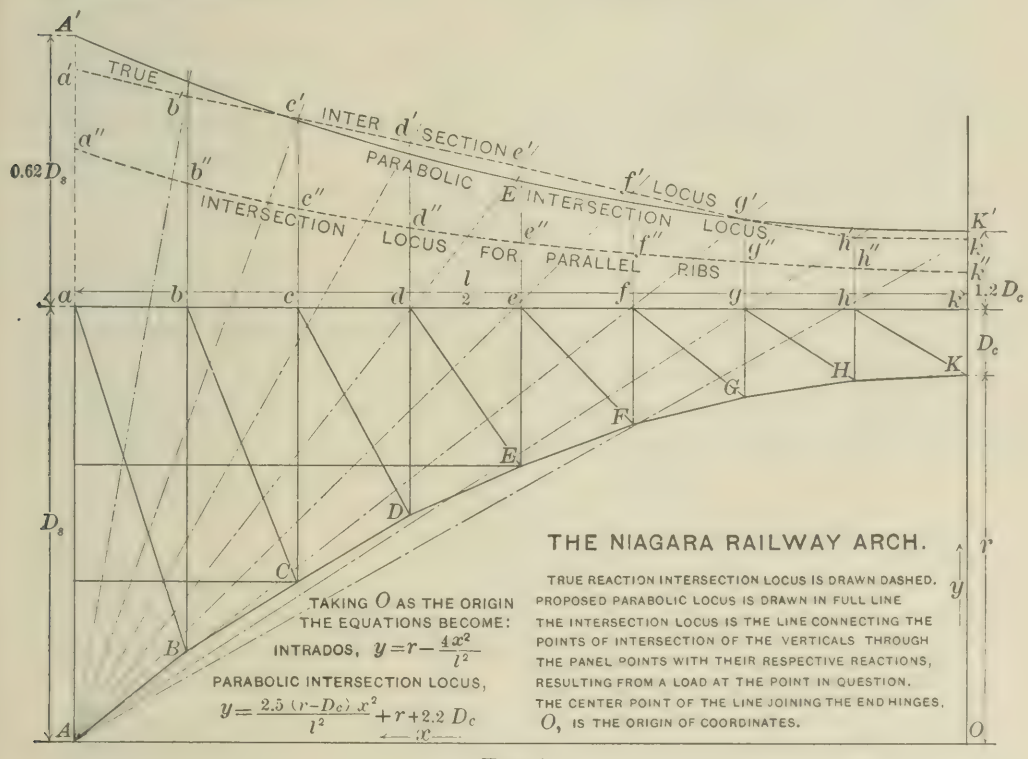

TRUE
INTER
SECTION
PARABOLIC
INTERSECTION
LOCUS
FOR
PARALLEL
RIBS
LOCUS
THE NIAGARA RAILWAY ARCH.
TRUE REACTION INTERSECTION LOCUS IS DRAWN DASHED.
PROPOSED PARABOLIC LOCUS IS DRAWN IN FULL LINE
THE INTERSECTION LOCUS IS THE LINE CONNECTING THE POINTS OF INTERSECTION OF THE VERTICALS THROUGH THE PANEL POINTS WITH THEIR RESPECTIVE REACTIONS, RESULTING FROM A LOAD AT THE POINT IN QUESTION.
THE CENTER POINT OF THE LINE JOINING THE END HINGES, O, IS THE ORIGIN OF COORDINATES.
TAKING O AS THE ORIGIN THE EQUATIONS BECOME:
INTRADOS, $y = r - \frac{4x^2}{l^2}$
PARABOLIC INTERSECTION LOCUS, $y = \frac{2.5\,(r - D_c)\,x^2}{l^2} + r + 2.2\,D_c$
0.62 D_s
D_s
D_c
1.2 D_c
r
y
x

FIG. 15.

The equation of the parabolic locus here used for a full spandrel arch, referred to the same origin, is

$$y = \frac{2.5\,(r - D_c)\,x^2}{l^2} + r + 2.2\,D_c \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

The full-line curve, A', E', K' is this locus. It will be seen that if reaction lines were drawn through the springings to the locus, a'', b'' ..... k'', these would all have greater horizontal components than those drawn to either of the others. That is to say, the horizontal components of the reactions for an arch of constant depth (but whose

* See *Modern Framed Structures*, p. 210; Eq. (19); also, the works of Greene and Du Bois.

Mr. Johnson. moment of inertia increases with the secant of the angle with the horizontal from the crown to the springings, this condition having been assumed in the derivation of this locus), are all greater than the corresponding horizontal components for a full-spandrel arch; but the difference between actual stresses in a full-spandrel arch and the stresses which would result from using the locus a'', b'', ... k'' would be less than 5% on the average. That is to say, if the reactions were assumed to be the same for a full-spandrel two-hinged arch as they are for such an arch of constant depth, or having parallel ribs, the error involved could not exceed 5% on any member.

It is evident, however, that all full-spandrel arches are much more nearly like each other than they are like arches having parallel ribs, and hence it must become evident that in assuming the Niagara reaction-intersection locus it will serve for the solution of all full-spandrel arches, and the error involved must be very much less than 5 per cent. Even if the approximate parabolic locus here found be used, the resulting error would certainly never be as much as 2%, and would probably always fall inside of 1 per cent.

When it is considered that other functions of the problem cannot be evaluated nearer than 50%, and that to cover these great uncertainties a factor of safety of 300% or 400% is inserted, is it not very irrational to expend great labor in trying to compute sections for assumed loads to the nearest tenth of 1 per cent.?

A somewhat closer approximation to the true locus may be found by using a hyperbola having the equation

$$y = \sqrt{\frac{8}{3}\left(\frac{x}{l}\right)^2 (Q^2 - K^2) + K^2}$$

where $Q = 1.35\, r + 1.85\, D_c$

and $K = r + 2.2\, D_c$

This curve coincides at the center with the parabola here used, but drops a little at the ends.

These reflections, and this approximate method of computing these structures, are submitted to the good judgment of practicing engineers, rather than to the equally worthy members of the Society whose principal business is to apply exact theories to very definite but assumed problems.

Mr. Braune. G. M. Braune, Jun. Am. Soc. C. E.—The writer fully agrees with the author in his praise of the steel arch. In beauty, the steel arch is entitled to first place among the different types of bridges. Were it not for that statically indeterminate force—the horizontal thrust—it would, no doubt, appear more frequently in America.

The following equations for determining the horizontal thrust for the two-hinged arch with web members were taken by the writer from the lectures of the late Dr. Fraenkel, of the Technical High School of

Dresden, Saxony, and from Mueller-Breslau's Graphical Statics.* which work treats of the arch very fully, and in such a manner that the theory is easily understood by the reader. Mr. Braune.

If S represents a force working in the axis of a member $m-m$ whose length is u; $\xrightarrow[m]{u}\ \xrightarrow[m]{S}$ a certain amount of work will be performed, which (assuming that the material of which the member is composed possesses the virtue of elasticity) will be equal to

$$A = \frac{S^2\, u}{2\, E\, F} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

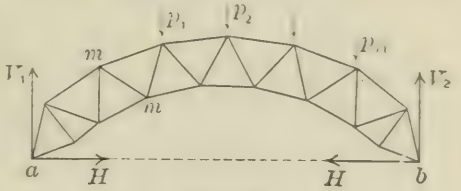


FIG. 16.

in which E denotes the modulus of elasticity, and F the cross-section of the member.

The work of a whole system, when forces act in the different members, would be:

$$A = \Sigma \frac{S^2\, u}{2\, E\, F} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (2)$$

Let the forces $p_1\, p_2$ ——— p_m act on the arch a–b; the reactions resulting from these forces may be resolved into V_1, V_2 and H. Let S be the stress in the member m—m, then $S = Q + s\, H$ may be written, in which Q is the stress in the member when $H = 0$, or when the arch acts as a simple beam, and s is the stress when $H = 1$.

Assume that the abutments are movable, and represent the amount of movement by $\Delta\, l$,

then $$-\Delta\, l = \frac{d\, A}{d\, H} \text{ (Castigliano)} \ldots\ldots\ldots\ldots\ldots\ (3)$$

in which A equals the work performed in the whole system, and H is equal to the horizontal thrust.

It has been shown that

$$A = \Sigma \frac{S^2\, u}{2\, E\, F},$$

so that by differentiating, and substituting in (3), there results:

$$-\Delta\, l = \Sigma \frac{S\, u}{E\, F} \frac{d\, S}{d\, H} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ (4)$$

From $S = Q + s\, H$, is obtained $\frac{d\, S}{d\, H} = s$; and substituting these values in (4):

$$-\Delta\, l = \Sigma \frac{(Q + s\, H)\, u\, s}{E\, F},$$

and hence

$$H = \frac{\Sigma \frac{Q\, u\, s}{F} + E\, \Delta\, l}{\Sigma \frac{s^2\, u}{F}};$$

* "Statik der Baukonstruction," by Mueller-Breslau.

Mr. Braune. or, when the abutments are regarded as fixed,

$$\Delta l = 0 \left\{ \quad H = \frac{\Sigma \frac{Q\ u\ s}{F}}{\Sigma \frac{s^2\ u}{F}} \right. \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Q = stress in member when $H = 0$.
u = length of member.
s = stress in member, when $H = 1$.
F = cross-section of each member.

The horizontal thrust resulting from the change in temperature may be found in the following manner.

If for each member of the arch truss the change in temperature is the same and is equal to t°, and were the reactions removed, the arch would expand and assume a shape similar to its original form, and the line of horizontal thrust would lengthen itself, $a\ l\ t^\circ$, l being the span, a the coefficient of expansion, and t° the difference in temperature. Now, the abutments would prevent this expansion through two equal forces H_t working in the direction of the line of horizontal thrust.

Now, $$-\Delta l = \frac{d\ A}{d\ H_t},$$

therefore $$-a\ l\ t^\circ = \frac{d\ A}{d\ H_t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Let S be the stress working in any member of the truss caused by H_t, then $S = H_t\ s$, and $\frac{d\ S}{d\ H_t} = s$ (and s = stress for $H_t = 1$).

It has been shown that $d\ A = \Sigma \frac{S\ d\ S\ u}{E\ F}$; substituting these values in equation (6) there results:

$$-a\ l\ t^\circ = \Sigma \frac{S\ u}{E\ F} \frac{d\ S}{d\ H_t} = H_t\ \Sigma\ \frac{s\ u}{E\ F}\ s$$

and, $$H_t = \pm \frac{E\ a\ t^\circ\ l}{\Sigma \frac{s^2\ u}{F}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which
E = the modulus of elasticity,
a = the coefficient of expansion,
t° = difference in temperature,
l = the length of span,
s = stress when $H = l$,
u = length of member,
F = cross-section of member.

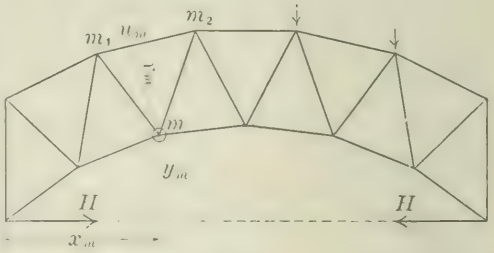


FIG. 17.

By disregarding the change in form (Formaenderung) of the web members, when forces act upon the truss,* equations (5) and (7) may be simplified in the following manner:

* Mueller-Breslau : Zeitschrift der A. & I. Verein, 1884, p. 581.

Mr. Braune.

Let u_m = length of member m_1—m_2.
r_m = perpendicular from m to u_m,
y_m = ordinate of panel point m,
F_m = section of u_m,
F_c = the most frequently occurring section.

$$H = \frac{\Sigma \frac{Q\ s\ u}{F}\ F_c}{\Sigma \frac{s^2\ u}{F}\ F_c} \qquad (5^a)$$

$$H_t = \frac{E\ a\ t^\circ\ l\ F_c}{\Sigma \frac{s^2\ u}{F}\ F_c} \qquad (7^a)$$

Let M_m be the moment of all the external forces about the point m and M^1_m the moment about the point m of a simple beam, then

$$M_m = M_m^{\,1} - H\ y_m,$$

and the stress in the member is:

$$S_m = \frac{M_m}{r_m},$$

or,
$$S_m = \left(\frac{M_m^{\,1}}{r_m} - \frac{H\ y_m}{r_m}\right).$$

It has been shown that $S_m = (Q_m + s_m\ H)$, in which Q = stress when $H = 0$, S = stress when $H = l$.

By comparing these two equations, it is found that

$$Q_m = + \frac{M_m^{\,1}}{r_m}; \text{ and } s_m = - \frac{y_m}{r_m};$$

hence,

$$Q_m\ s_m\ u_m \frac{F_c}{F_m} = + \frac{M_m^{\,1}\ s_m\ u_m}{r_m} \frac{F_c}{F_m} = \frac{M_m^{\,1}\ y_m\ u_m}{r^2_m} \frac{F_c}{F_m};$$

and,

$$S^2_m\ u_m \frac{F_c}{F_m} = - \frac{y_m\ s_m\ u_m}{r_m} \frac{F_c}{F_m} = - \frac{y_m^2\ u_m}{r^2_m} \frac{F_c}{F_m}$$

and, by substituting in equations (5^a) and (7^a),

$$H = \frac{\Sigma\ M_m^{\,1}\ w_m}{\Sigma\ Z_m} \qquad (5^b)$$

and,
$$H_t = \frac{E\ a\ t^\circ\ l\ F_c}{\Sigma\ Z_m} \qquad (7^b)$$

$$W_m = \frac{y_m\ u_m}{r^2_m} \frac{F_c}{F_m}$$

in which
$$Z_m = - y_m\ w_m = - \frac{y^2_m\ u_m}{r^2_m} \frac{F_c}{F_m}.$$

Mr Braune. Now let a force P act at a point $x = a$ of the truss $l - l$, then the moment about a panel-point to the left of P will be:

$$M^1{}_m = \frac{P\,(l-a)}{l}\,x_m,$$

and for a panel-point to the right of P:

$$M^1{}_m = \frac{P\,a}{l}\,x^1{}_m.$$

If all the moments of the panel-points to the right and left of P are found and substituted in equation (5'), there obtains:

$$H = P\,\frac{\frac{l-a}{l}\,\Sigma^{a}\,w_m\,x_m + \frac{a}{l}\,\Sigma^{l-a}\,w_m\,x^1{}_m}{\Sigma\,Z_m} \qquad \ldots\ldots\ldots \quad (8)$$

Now if the values of w_m are multiplied by the force $P = l$, and the moments of these forces about the point $x = a$ are found (allowing the forces w_m to act, of course, in the respective panel-points for which they were calculated) there results:

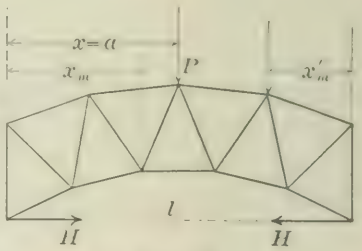


FIG. 18.

$$M\,w = \frac{l-a}{l}\,\Sigma^{a}\,w_m\,x_m + \frac{a}{l}\,\Sigma^{l-a}\,w_m\,x^1{}_m.$$

The right side of this equation is identical with the numerator of equation (8), and, by substituting, it is found that,

$$\left.\begin{aligned} w_m &= -\frac{Y_m\,u_m}{r^2{}_m}\,\frac{F_c}{F_m} \\ Z_m &= -\,Y_m\,w_m \end{aligned}\right\} \quad H = \frac{M\,w_m}{\Sigma\,Z_m} \qquad \ldots\ldots\ldots\ldots \quad (9)$$

Therefore, to find the horizontal thrust which a force $P = l$ exerts at a point $x = a$ of the arch truss, the values $W_m = \frac{Y_m\,u_m}{r^2{}_m}\,\frac{F_c}{F_m}$ must be calculated and multiplied by $P = l$, and the bending moment of these forces (allowing them to act at the respective panel points, for which they were figured) found about the point $x = a$.

FIG. 19.

These equations may, at first, appear rather tedious, but after using them a few times they are solved very readily.

By putting $\frac{F_c}{F_m} = 1$, the equations are much simplified, and the results thus obtained (Mueller-Breslau) are very satisfactory.

The value F_c of the equation (7') may be calculated as follows: After having found the stress acting in a member T due to the live and dead loads, an equation,

$$S\,F_c = Q_L + Q_D + Q_t,$$

may be written, in which S = the stress allowed per square inch, Mr. Braune.

Q_L = the stress resulting from live load,

Q_D = " " " dead "

Q_t = " " " change of the temperature.

Having calculated the expression $\frac{E\,a\,t^{\circ}\,l}{\Sigma\,Z_m}$ from (7'), let it be represented by B;

then, $$S\,F_c = -\,Q_L + Q_D + \frac{h}{r}\,B\,F_c,$$

and, $$F_c = \frac{Q_L - Q_D}{s - B\,\frac{h}{r}}.$$

For the case where the top chord is horizontal, and the panel lengths are equal, there may be written:

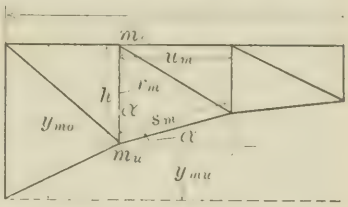


FIG. 20.

$$W_{mo} = \frac{S_m\,y_{mo}}{r^2_m}\,\frac{F_c}{F_m}$$

$$= \frac{U_m}{\cos\alpha}\;y_{mo}\;\frac{1}{h^2\cos^2\alpha}\;\frac{F_c}{F_m}$$

$$= \frac{Y_{mo}\,U_m}{\cos^3\alpha\;h^2}\,\frac{F_c}{F_m}$$

and $$W_{mu} = \frac{y_{mu}\,U_m}{h^2}\,\frac{F_c}{F_m}$$

combining these two

$$W_{mu} + U_{mo} = W_m = \frac{U_m}{h^2}\left(y_{mu}\,\frac{F_c}{F_m} \times y_{mo}\,\frac{F_c}{F_m}\,\frac{1}{\cos^3\alpha}\right)$$

For all practical purposes, since the panel-lengths are equal, these equations may be written:

by putting $$\left(\frac{F_c}{F_m} = \frac{F_c}{F_m}\,\frac{l}{\cos^3\alpha} = l,\right)$$

$$W_m = \frac{Y_{mu} + Y_{mo}}{h^2};\quad Z_m = \frac{Y^2_{mu} + Y^2_{mo}}{h^2}$$

R. S. BUCK, M. Am. Soc. C. E.—In response to several requests, the author gives herein the outline of the method of calculation used in this design, and, in order to make clearer the essential points involved, has added some details and explanations not given in Professor Greene's book. Mr. R. S. Buck.

The removal of the center hinge eliminates the point that admits of an easy graphical determination of the proper force polygon for any loading, and the equation of condition for invariability of span is not applicable. This equation,

$$\Sigma\,\frac{E\,F\,.\,D\,E}{E\,I} = 0,$$

becomes $$\frac{1}{E}\,\Sigma\,\frac{E\,F\,.\,D\,E}{I} = 0$$

Mr R. S. Buck. when I becomes a variable, as it does with the spandrel-braced arch with horizontal top chord.

In this equation $E\ F$ is the vertical intercept between the force polygon for any loading and the axis of the rib; and $D\ E$ is the ordinate to the same point on this axis (see Fig. 21). E is the modulus of elasticity of the material in the arch, which is usually constant. I is the moment of inertia of the truss, which varies rapidly, and cannot be determined at any point until the sections are known. It must, therefore, come within the sign of summation, which renders it impracticable to apply the above formula.

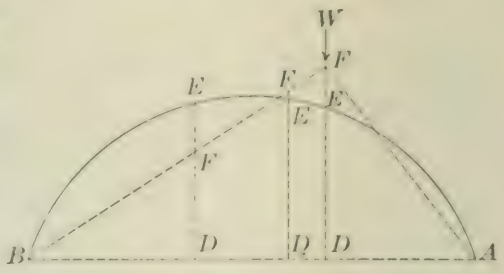


FIG. 21.

The principle involved in the Clerk-Maxwell method is the following: If one end of an arch is fixed and the other end free to move, and if any member of the truss is lengthened or shortened under stress, the horizontal movement of the free end of the arch due to this change of length will be proportional to the horizontal thrust at this point, if the movement were prevented. In other words, internal work of strain equals external work of resistance.

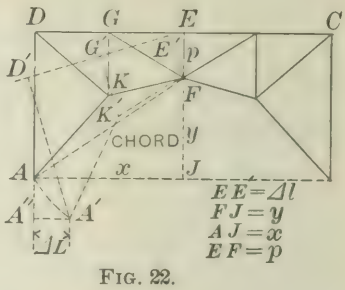


FIG. 22.

To demonstrate more fully (see Fig. 22):

Let T = stress in any member $G\ E$, no other member being under stress.

" A = area of cross-section of $G\ E$,

" E = modulus of elasticity "

" l = length of $G\ E$,

" Δl = elongation or compression of $G\ E$ under stress T.

Then $$\Delta l = \frac{T}{E\ A} \times l \ldots\ldots\ldots \quad (1)$$

Now, if we consider the position of the arch on the right of $G\ E$ to be held rigidly and this member lengthened or shortened an amount Δl, for a slight distortion motion of the free end of the arch, F can be considered as taking place about F as a center, this being the center of moments of the piece in question. We then have

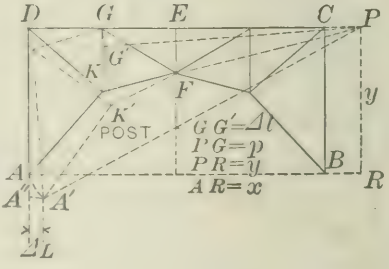


FIG. 23.

$$A\,A' : E\,E' :: A\,F : E\,F \therefore A\,A' = \frac{E\,E' \times A\,F}{E\,F} = \frac{\Delta l \times A\,F}{p}$$

Again $$A\,A' : \Delta\,L :: A\,F : F\,J \therefore \Delta\,L = \frac{A\,A' \times F\,J}{A\,F}$$

$$= \frac{y}{p}\,\Delta l \ldots\ldots\ldots\ldots\ldots \quad (2)$$

In other words, the horizontal component of the movement of the free end of the arch is to the change of length of the member stressed as the vertical ordinate of the center of moments of the member is to the perpendicular to the member dropped from the center of moments.

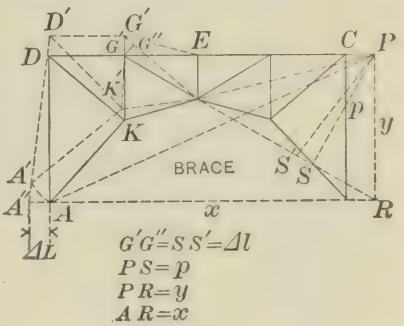


Fig. 24.

This is more readily apparent in the case of the top chord than of the vertical and inclined members. The distortion of the truss for change of length of post, brace and rib is given in Figs. 23, 24 and 25; and the operations of obtaining the values of ΔL for these cases are as follows:

Post.—$$A\,A' : G\,G' :: A\,P : P\,G \therefore A\,A' = \frac{G\,G' \times A\,P}{P\,G}$$

$$A\,A' : \Delta\,L :: A\,P : P\,R \therefore \Delta\,L = \frac{P\,R \times A\,A'}{A\,P} = \frac{\Delta l \times A\,P}{p}$$

$$= \frac{y}{p}\,\Delta l$$

Brace.—$$A\,A' : S\,S' :: A\,P : P\,S \therefore A\,A' = \frac{\Delta l \times A\,A'}{P}$$

$$A\,A' : \Delta\,L :: A\,P : P\,R \therefore \Delta\,L = \frac{y \times A\,A'}{A\,P} = \frac{y}{p}\,\Delta l$$

Rib.—$$A\,A' : K'\,K'' :: A\,G : G\,S \therefore A\,A' = \frac{\Delta l \times A\,A'}{P}$$

$$A\,A' : \Delta\,L :: A\,G : G\,R \therefore \Delta\,L = \frac{y \times A\,A'}{A\,G}$$

$$= \frac{y}{p}\,\Delta l$$

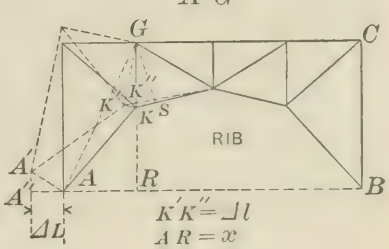


Fig. 25.

Now substituting in equation (2) the value of Δl given in equation (1) we get,

$$\Delta\,L = \frac{T}{E\,A}\,\frac{l\,y}{p} \ldots\ldots\ldots\ldots\ldots \quad (3)$$

If the arch be acted upon by two forces H and V (Fig. 26), by the method of moments, the stress in any member as $G\,F$ can be obtained.

$$T = \frac{H \times P\,R + V \times A\,R}{P\,S} = \frac{H\,y + P\,x}{p} \ldots\ldots\ldots \quad (4)$$

Mr. R. S. Buck. Substituting this value of T in equation (3), there results

$$\Delta L = \frac{H y^2 + P x y}{p^2} \frac{l}{E A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Now, this value of ΔL can be calculated for the change of length of each member in the arch, and the error of considering each member as changing its length independently of the changes in the other members is inconsiderable for slight distortions.

The total moment, therefore, is

$$\Sigma \Delta L = H \Sigma \frac{y^2}{p^2} \frac{l}{E A} + P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A} \ldots\ldots (6)$$

where P_1 is the vertical end reaction at the left, and P_2 the same at the right.

There is no actual movement of the point A, therefore, $\Sigma \Delta L = 0$, and we have

$$H \Sigma \frac{y^2}{p^2} \frac{l}{E A} + P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A} = 0 \ldots\ldots (7)$$

$$H = \frac{P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A}}{\Sigma \frac{y^2}{p^2} \frac{l}{E A}} \quad \ldots\ldots\ldots\ldots (8)$$

which is the general value for the horizontal thrust due to a load at any point on the arch.

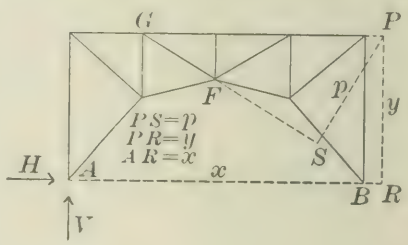


FIG. 26.

The first expression in the numerator is for those members to the left of the load and the second for those to the right of the load. The denominator covers all the members in the truss.

In this expression for the value of H, E is constant, and can therefore be cancelled out.

In order to obtain trial values of H and from these to obtain trial sections, it is necessary first to consider the sections of all members as being the same.

Thus A is also constant and cancels out.

Then, with values of A proportionate to the unit load stresses, the values of H can be properly corrected by giving due consideration to the variability of A.

With A constant, equation (8) becomes

$$H = \frac{P_1 \Sigma \frac{x y}{p^2} l + P_2 \Sigma \frac{x y}{p^2} l}{\Sigma \frac{y^2}{p^2} l} \quad \ldots\ldots\ldots\ldots (8^a)$$

As can be readily seen, $\frac{x}{p}$ is simply an expression for the stress in any

member due to a vertical end reaction of unity, and $\frac{y}{p}$ the same for a horizontal thrust of unity. Mr. R. S. Buck.

Therefore, the first steps in the solution of the above expression for H is to ascertian these stresses in all members. These can be obtained by any of the ordinary methods, preferably perhaps by means of moments for the chord members, and graphically for posts, braces and ribs.

A combination of the two methods serves as a very convenient check.

With values of H for load unity on each panel point, the unit stresses of all the members can be obtained.

The values of H for load unity on each panel-point are as follows:

	Preliminary. *A Constant.*	Final. *A Variable.*
Load on 1	.15554	.16459
" " 2	.29824	.31400
" " 3	.43439	.45411
" " 4	.56250	.58499
" " 5	.67596	.70175
" " 6	.77945	.79974
" " 7	.85051	.86702
" " 8	.87614	.88540
(16 Panels Loaded)	8.39652	8.65840

The results show only a variation of from 1 to 6%, and therefore a second approximation is unnecessary.

There is marked similarity between this method of Clerk-Maxwell and that given by Mr. Braune as taken from the German authorities. Applied to the truss given in Professor Greene's book on arches to illustrate the Clerk-Maxwell method, Mr. Braune's equation (5) gives values of H from 1 to 9% smaller.

Professor Johnson's views concerning the sufficiency of approximate methods in such designs as require so much work to compute are unquestionably sound. His deductions are certainly serviceable. One or two applications, however, of the more tedious methods show numerous short cuts within the limits of theoretically correct treatment that deprive them of much of their capacity for consuming time and grey matter.

Mr. Schaub's desire to launch a stringent criticism rather clouds his observation of facts.

The full amount of dead load was not imposed on the Niagara Railway Arch when the adjustment was made by means of the compression toggle. None of the upper floor system, and only part of the lower floor system, was placed at the time the adjustment was made. In fact, when due consideration was given to temperature conditions, only

Mr. R. S. Buck. 350 tons had to be imposed in both top chords to secure proper adjustment, instead of 850 tons, as imagined by Mr. Schaub.

If Mr. Schaub examines the toggle a little more closely he will find that it is capable of much more push than may at first appear. The bolts are 2¼ ins. in diameter, and with ordinary wrenches the nuts could be drawn up to produce a tension of 4 tons in each bolt. This means 16 tons applied to both sides of a double toggle, or 32 tons effective lateral pressure. The angle of inclination of the toggle proper multiplies this by 8, and gives an available pressure of 256 tons for each toggle, or 516 tons for both chords to take care of 350 tons needed.

So far from having practically nothing in the way of dead load stress in the top chord of the center panel, there is a reasonably close approximation to the 350 tons due at the time of adjustment, and the balance due to subsequent loading is taken care of in the regular way.

The openings at the center of the top chords were increased by means of the toggles from ¼ in. to 1 in., and the line of camber, according to close level observations, raised over the entire span in a manner to indicate closer conformity to a correct distribution of stress.

The author takes exception to the proposition that the two-hinged arch is incapable of proper adjustment, and insists that such adjustment is not only possible, but easily applicable. He would, however, recommend a more sensitive means of measuring the stress applied at the center, such as was used in the case of the Niagara Falls and Clifton Arch, viz., hydraulic jacks with gauges attached. The device, however, used in the case of the Railway Arch was designed to meet an unexpected contingency, and was not, as in the latter case, carefully designed beforehand.

Mr. Lewinson's proposition that the form of end bearing here adopted is improper, on account of the theoretical center of the end bearing being 4 ft. away from the bearing face of the joint, is readily answered.

Eccentricity and consequent appreciable unfair distribution of the pressure in the rollers are not possible so long as the line of pressure does not depart considerably from the center of the roller bearing. The extreme range of the line of pressure is only 24° 10′, and the usual range not more than a quarter of this; therefore, the rollers are not, as Mr. Lewinson imagines, very seriously abused. No bending can occur at this joint unless the line of pressure actually falls outside of the roller bearing, which is manifestly impossible.

The case there, in effect, is simply that of a pin 8 ft. in diameter with roller bearings.

The rollers are easily accessible and can be cleaned readily, therefore danger of their becoming ineffective from rust is not at all serious. As far as can be seen, there is not the least indication that there is any unfair distribution of stress on the rollers.

Mr. R. S. Buck.

Replying to Mr. Lindenthal's request for further information on certain points, the dead load concentrations and the allowable stresses for the truss have been added to the stress diagram.

The method adopted to secure the deflection under a uniform load of 10 000 lbs. per lineal foot was to compute the shortening of the rib and the consequent reduction of versed sine under this load. This dealt with the rib members only, and gave $2\frac{5}{16}$ ins. as the deflection. By using the "Pull over E" formula, $2\frac{1}{8}$ ins. are obtained as the deflection.

Theoretically, therefore, under a load of 6 500 lbs. per lineal foot, the deflection should be $1\frac{7}{16}$ ins. instead of $1\frac{3}{16}$ in. as actually obtained under this load. The difference, $\frac{5}{8}$ in., is proportionally considerable, but the actual amount of deflection is too small, and existing conditions too far removed from those necessarily assumed in theoretical treatment, to use the deflection as a close check on the stresses. All that can be accomplished is to see whether or not the deflection comes within the theoretical limit.

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No. 837.

MARINE WOOD-BORERS.

By Charles H. Snow, M. Am. Soc. C. E.

Presented at the Annual Convention, July, 1898.

WITH DISCUSSION.

The forms of marine life which attack wood are much more numerous than is commonly supposed. The wood-borer is for the most part concealed from view while at work. It is in consequence frequently dissociated from the subsequently discovered results of its labors.

The Teredo Navalis, being common in Europe, where these forms of life were first studied, has become better known than its companions. Nearly every kind of boring found in wood which has been in sea water is in consequence very generally attributed to this animal. The Teredo Navalis is worthy of all the attention it receives, being important of itself, and also standing as a representative of its family. That there are other species of teredo than the navalis, and other wood-borers than the teredo, must, however, not be forgotten.

The destruction accomplished by wood-borers is very great. The cost of wood destroyed, of replacements necessitated by failures, and of protecting wood so that it will not fail, aggregate a very large amount. The influence of these animals has not always been con-

fined to direct cost. It is said that their presence in abnormal quantities has affected the prosperity of cities. The weakening of the pile foundations and other woodwork of the dikes of Holland once threatened the very safety of that country.

The Teredo Navalis has been observed in Europe for nearly two centuries. The Dutch, from necessity, excelled in the early study of its habits, and the reputation thus established has continued to the present time. Much attention has been devoted to this and other forms of marine wood-borers by American naturalists, among whom the names of A. E. Verrill, of Yale University; S. I. Smith, of the National Fish Commission; W. H. Dall, of the Smithsonian Institution, and lately C. O. Siegerfoos, of the Johns Hopkins Department of Biology, should be mentioned.

The Teredo.

The teredo is a very ancient form of life, fossil remains having been found both in England and America. It was known to the Ancients, and is mentioned in the writings of Pliny and of Ovid. It was observed in modern times about the year 1730, when, as has been stated, it threatened the woodwork of the dikes of Holland.

The Teredo Navalis is often referred to as the Ship Worm, although it is a mollusk. The name is convenient, because the navalis and other teredos resemble worms in appearance, but if it is to be used it should include all forms of the teredo, and not be confined to the species navalis, in which case the names Teredo and Ship Worm rather than the names Teredo Navalis and Ship Worm would be synonymous.

Seven species of the teredo have been identified as existing in the United States. They are the Teredo Navalis (Linn.), the Teredo Norvegica (Spengler), the Teredo Dilatata (Simpson), the Teredo Megotara (Hanley), the Teredo Thompsono (Tryon), the Xylophaga Dorsalis (Forbes and Hanley), and the Xylotrya Fimbriata (Jeffreys). These varieties are similar in their principal characteristics. A description of the teredo may profitably be preceded by a mention of two of the more familiar forms of life which resemble it.

The long or soft shell clam (Myra), is familiar to all. The prominent characteristic is a long worm-like neck, which is out of all proportion to the shell which covers the softer parts. This long neck

contains two tubes or syphons, through one of which the animal receives its sustenance, the other being used to expel the water from which the microscopic life has been withdrawn.

Another form of the same worm-like structure is the razor clam (Solenidæ), whose shape is more nearly conformed to by the covering shell. It possesses a powerful club-shaped foot or sucker, which is used as an auger to penetrate the sand in which it resides. The special features to which attention is called are the long worm-like syphon structure in both the long and razor clam, and the foot or sucker of the latter. A general idea of the teredo can be gained by imagining a soft clam with an unusually long neck and a very small shell, or a razor clam with its shell cut away so as to expose nearly all of its length. The long clam, the razor clam and the teredo are all true mollusks, and only resemble worms in that parts of their bodies are long and round. The shell of the adult teredo covers but a small proportion of its slender worm-like body.

FIG. 1.

The body of the teredo (Fig. 1), which in substance resembles that of the oyster, is long, slender, smooth, soft and gray, tapering somewhat toward the outer or posterior end *s*, where it is marked by a wrinkled collar, shown at *C*. This is the end of the animal which is nearest to the entrance of the burrow. Although the collar attaches the animal to the sides of the boring which it has formed, yet the teredo is free to move to either side of it. The two little horns shown at *s* are the extremities of the tubes or syphons which have been described in connection with the long and razor clams. These little tubes may be extended out through the opening in the wood. They are the portions of the animal which are first noticed by the observer (Plate XIII).

The two little shelly plates, *p*, at the base of the syphon prolongations, are called pallets. They are slightly curved at the top so that they can enfold the syphons. When the syphons are withdrawn into the burrow, the pallets are contracted over them so as to protect these soft tubes from the enemy. The pallet shells are frequently confused with the boring shells, which are at the other end of the body.

TEREDO AT WORK.—LIFE SIZE.

The principal or boring shells *B*, Fig. 2, which are open both at the top and bottom, are small, and very beautifully formed. The opening at the top permits the emergence of the body, and the foot or sucker *f* passes through the opening at the bottom. A calcareous substance is invariably deposited by the teredo upon the newly cut surface of the wooden tunnel, forming an enameled lining through which the animal can glide backward or forward as it contracts and expands. This shell tube is a separate shell structure, distinct from the pallets and from the boring shell.

It will be seen that the distinctive features of the teredo are the body, the collar, the syphons, the pallets, the boring shell, the foot and the lining shell. These several members and their processes will be considered separately.

The Body.—The body of the teredo resembles that of a long worm, without the articulations. In the young animal it is so transparent that some of the interior organs, such as the heart and the ovary, may be observed through it. The heart consists of two auricles and a ventricle. The pulsations, which may be readily counted, are irregular, the rate being about four or five per minute. The blood is a transparent, colorless fluid. Many of the important organs, as the mouth, the palpi, the liver and the foot, are enclosed in the boring shell at the further extremity of the animal. The gills are located for the most part at the outside of the shell, and are very interesting. They are long and narrow, usually reddish brown in color, and perform the important office of sheltering the eggs and embryo. The nervous system is well developed, and consists of filaments and ganglions connecting the mouth, the branchiæ, the foot, the collar and the syphons. The stomach is not distinguished by any peculiarity, but there is a well-developed intestine. The great length of the body is due to the elongation of the syphons or breathing tubes.*

The Collar.—The collar C, Fig. 1, is a muscular, wrinkled membrane which extends entirely around the posterior portion of the animal, and forms a connection between the teredo and the shelly lining of its tunnel. This is the only place at which the teredo is not free and separated from its surroundings. The collar fills the place between the teredo and the circumference of its tunnel. Water cannot pass through the orifice of the tunnel, save as it is controlled by the

*Johns Hopkins Circular, 1896.

syphons. The collar contains several well-defined muscles, and these act upon the pallets which are pulled down over the syphons in such a manner as to close the entrance to the tube when the extremities of the syphons are drawn into the burrow.

The Syphons.—The syphons are the two principal organs, and extend throughout the greater length of the body. One of these tubes conveys the oxygen, water and infusorial food to the vital processes of the animal; the other conveys the exhausted water, the excretions, the débris from the excavation and the eggs to the free water without. The outer structures of the syphons are united while they remain in the body, but signs of divergence are seen as they emerge from between the pallets. They continue united for a little and are then separated into two distinct tubes, as indicated in Plate XIII. These divergent extremities pass backward and forward through the orifice in the wood. They constitute the only part of the animal which can be seen from the outside of the wood. The extremities often stretch for some distance out through the minute opening to the cell, and are sometimes mistaken for the entire animal.

The extremities, which usually appear to be about equal in length as seen from the exterior of the wood, are yellowish or white in tint, but are sometimes speckled with reddish brown. The longer or incurrent extremity can be pushed out to a distance of 2 ins. or more, while the outcurrent throat remains at about half that distance.

The teredo is able to expand or contract these extremities at will, and when the conditions are favorable, they are extended through the orifice to their full length, and remain stationary or wave slowly backward and forward. The motion is sometimes confined to the extreme ends, while the greater portion of the extended extremity remains stationary. When the animal is alarmed, the syphons are withdrawn and pass down between the pallets into the tunnel; the pallets close over them and protect them from harm. The syphons are erected by means of a current of blood sent into them from the vessels within. When the water is warm, the animal is active and the syphons are extended out full length. They are withdrawn when the water becomes cold, and the teredo is entirely hidden. Plate XIII shows the syphons fully extended as they appeared after several consecutive days of warm weather. The extremities of the syphons must always be kept at the orifice of the wood. As the animal grows, the muscular

FIG. 1.
LINING SHELL, KUPLIUS GIGANTEA, 41 INS. FROM ENTRANCE.

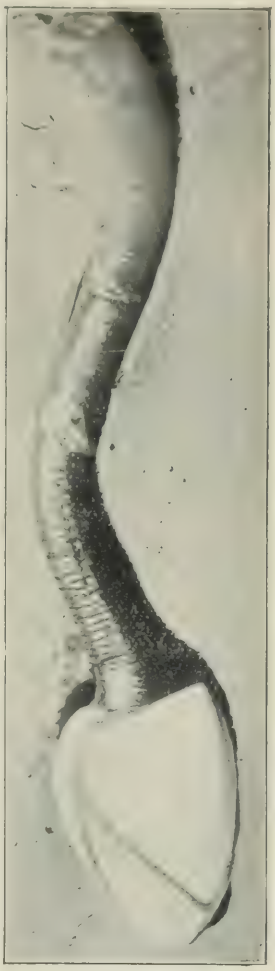

FIG. 3.—BARNACLE.

FIG. 2.
ENTRANCE, KUPLIUS GIGANTEA, SHOWING DIVISIONS FOR SYPHONS.

FIG. 5.
PALLETS.

FIG. 4.
YOUNG TEREDO.

ALL LIFE SIZE.

collar and the pallets recede from the entrance, so as to permit the extremities of the syphons to remain there.

The Pallets.—The two shelly plates near the orifice are called pallets (Fig. 5, Plate XIV). They are broad and flattened at the top and concentrated at the base. These long slender basal columns are connected with the muscles of the collar so that the pallets may be relaxed or contracted at will. They relax when the syphon extremities pass out between the crescents or horns, which will be seen at the top of the shells, and contract when the syphons are withdrawn. The pallets are then folded over so as to serve as an operculum to protect the soft tubes from enemies. The Xylotrya Fimbriata differs from the ordinary variety in that its pallets are long and oar-shaped. The stalk is slender, the blade is flattened on the inside and is convex externally. It consists of ten or twelve funnel-shaped segments set into one another, and having their margins projecting at the sides, so that the edges of the blade appear serrated.

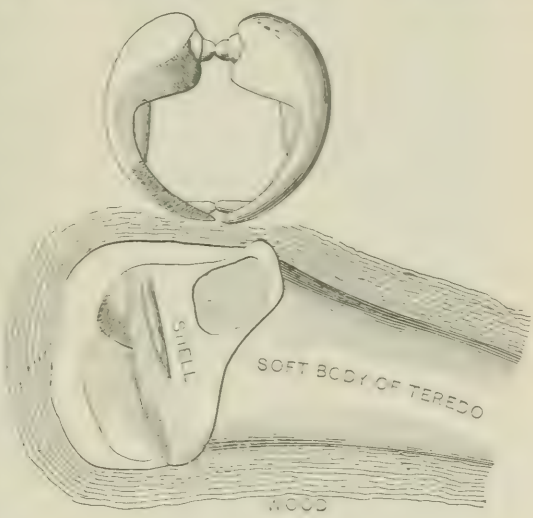

Fig. 2.

The Boring Shell.—The boring shell, which is nearly as long as it is broad, presents an irregularly triangular appearance when observed from the side. It may be best seen in Figs. 1 and 2. The two halves close tightly at the hinge and at the side opposite the hinge; the open space at the top being toward the main bulk of the animal, and the opening toward the extremity of the tunnel permitting the emergence of the foot or sucker. The shells of young animals are larger in proportion than those of old animals, as shown in Fig. 4, Plate XIV. When the animal is very young, it is for a short time entirely enclosed in the shell.

The Foot.—The foot, which in form resembles a pestle, is a short, stout, muscular organ, broadly truncated or rounded at the end, and so arranged that it can exert a powerful suction upon anything to which it is attached. This cupping action assists the shell in excavating to an extent which has probably not been understood.

The Lining Shell.—This shell, which has been previously described, follows the tunnel until it finally terminates in a spherical cap, which may be seen to the left of Fig. 1, Plate XV. The adult animal occasionally shrinks, and a second cap or partition may be formed as he retreats, the space between the caps remaining unoccupied.

The early portion of the tunnel, which was cut by the young animal and is now occupied by the syphon extremities, is below the normal diameter. The lining shell at this point sometimes divides itself into two tubes, each containing one of the syphons. This structure is well shown in Fig. 2, Plate XIV. It has been stated that the extremities of the syphons must be kept at the orifice of the wood. The collar and pallets recede as the animal grows larger, thus forming a considerable space between the pallets and the orifice of the tunnel, which is filled with little rings of shell, whose sharp edges serve to protect the entrance. The slender syphons emerging from the pallets pass readily through the spaces at the centers of the rings.

The teredo can rarely advance for any time in a straight line, being forced to deviate therefrom so as to pass around obstacles, such as cracks, knots or the pre-existing tunnels of its companions. The tunnels may wind in and out and pass so close to one another as to occupy almost the entire content. This fact is well illustrated in Fig. 2, Plate XV. The thickness of the lining varies with the species. It is sometimes so thin and fragile that it becomes detached by the slightest shock. Many of the dried specimens exhibited in museums do not show the lining shell for this reason. The shell is sometimes very thick. The specimen shown in Fig 1, Plate XIV, exhibits an extreme thickness of half an inch. It was 41 ins. long and 3 ins. in diameter at the larger end. This species exists in the tropics, and does not penetrate wood, but forces its way into the sand, thus resembling the razor clam. The shells of this variety of teredo sometimes become covered with oysters or other mollusks. The lining presents a surface against which the soft body of the mollusk may press without injury. It seals the interior of the cell so that the water supply may be better controlled by the syphons.

The teredo rarely crosses a seam or joint in the wood, probably because it fears for the integrity of the lining. Specimens exhibiting the attempt of the teredo to cross a seam show that the shell has been much strengthened at the junction point. When the teredo arrives at

FIG. 1.—DISSECTION SHOWING LINING SHELL THROUGH WOOD. LIFE SIZE.

FIG. 2.—MASS OF LINING SHELLS. LIFE SIZE.

FIG. 3.—END VIEW OF FIG. 1. LIFE SIZE.

maturity, or whenever an insurmountable obstacle is encountered, it seals over the inner extremity of the lining. The growth of the animal, in length, is stopped in either case, and it is entirely surrounded by shell. There is a communication with the outer sea at the two syphon points only, and as the animal continues to live for some time under these conditions it is evident that its sustenance is derived from other sources than the wood.

Vital Processes.—The teredo resembles other bivalve mollusks in that it exists upon infusorial life. This food, together with the necessary amount of oxygen, is drawn in through the longer or incurrent syphon, and flows throughout the length of the animal until it reaches the mouth at the other extremity. The mouth, stomach and intestine are well developed and perform their usual offices. The oxygen is retained by the gills. The return current, beginning at the gills, removes the exhausted water, the excretions and the woody particles. These flow out through the animal and are ejected by the shorter or outcurrent syphon. The teredo does not devour wood; its form is such that dust and other débris have to pass through its body to the point of ejection. Where a teredo is watched for some time, small clouds of very fine dust may at length be observed puffed out from the orifice. The circulation through the syphons is continuous.

The teredo may live for a short time out of water. This fact explains its ability to attack wood between high and low water. The specimens which enter wood where it is exposed between the tides do not seem to be greatly hindered in their work.

Th. G. Hoech, M. Am. Soc. C. E., states* that teredos have been found alive in wood which had been removed from the water for two months. The circumstances are not stated, but it is probable that the wood was in bulk and remained moist in consequence.

Many of the logs in a cargo of Central American woods recently received in New York City, after a voyage of about two weeks, were found to be occupied by living teredos, which had gained entrance to the wood while it was waiting shipment in Southern waters. They were alive, strong, apparently healthy and able to resume their work if the wood should be again submerged. It is probable that considerable water was contained in the wood and hold of the vessel, yet the logs were certainly not submerged, and the fact remains, that these particular teredos survived the voyage and were so numerons as to

* Report on Woods Exempt from Attack by the Teredo.

emit a strong, disagreeable, fishy odor, which necessitated the removal of the logs from the yard.

R. Montfort, M. Am. Soc. C. E., states that teredos occupying standing piles were killed when these piles were encircled by loose iron jackets.* The teredos were in no wise injured by the jackets and the presumption is that the jackets were forced into the mud and that the teredos were destroyed by the muddy water. Where infected vessels are subjected to fresh water, the teredos usually live for about two weeks, so far as known.

When specimens of wood containing teredos are removed from sea water, the syphons relax after the vitality of the animal is gone. If the specimen be held upside down, the syphons then fall outward to a distance of 1 in. or more, according to the size of the animal within.

The Boring Apparatus.—While the animal is still very small, it settles upon the surface of the wood and almost immediately begins to clear away a place in which to burrow. A small pit is made by the edges of the valves of the shell, which come together on pivots shown in Fig. 2. The shells are controlled by powerful muscles acting so as to swing them backward and forward upon the pivots. Only a few of the teeth upon the shell are shown in Fig. 2, and these are exaggerated in size. When the posterior muscle contracts, the shell, with the teeth, is thrown outward and backward and rasps upon the surfaces of the wood. The process is assisted by the foot which emerges through the large blank space between the shells and performs a cupping action.

The teredo differs from many of the pholos tribe. It probably employs the inflated syphons and pallets to some unknown extent as a fulcrum. The lower portion of the animal has a large field of rotation, so that the various portions of the anterior end of the burrow can be grated away in turn. The large end of the burrow is, of course, slightly larger than the diameter of the shell. The principal work of excavation is accomplished by the shell. Many of the stone-borers, on the other hand, brace themselves by means of the shell which they open against the walls of their burrow and hold there as a fulcrum. The muscular foot emerges from the shell, and, assisted by the grit of previous borings, grinds at the stone. The wood-borers, so far as known, excavate with the shell, assisted by the foot. Xylophaga and teredina probably bore like the teredos.

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 227.

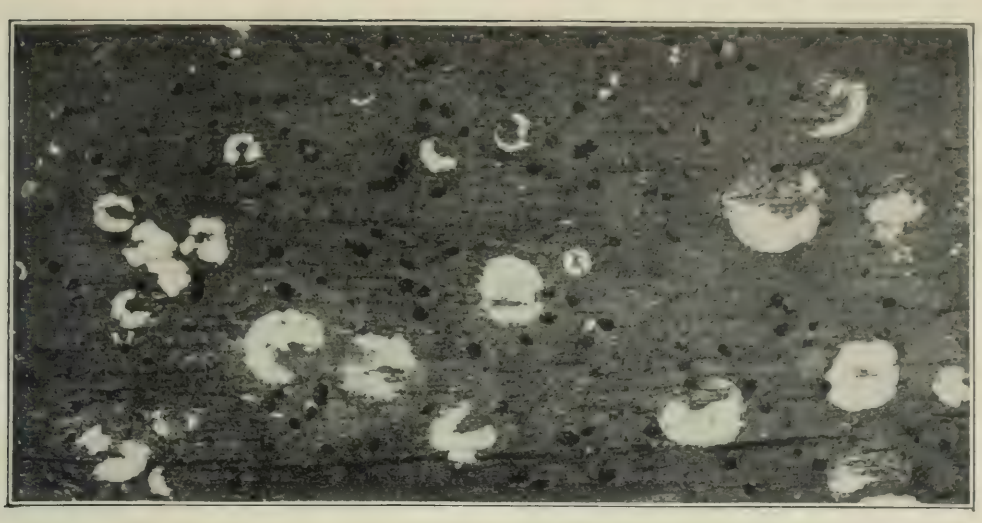

FIG. 1.—SURFACE OF WOOD OCCUPIED BY TEREDOS. LIFE SIZE.

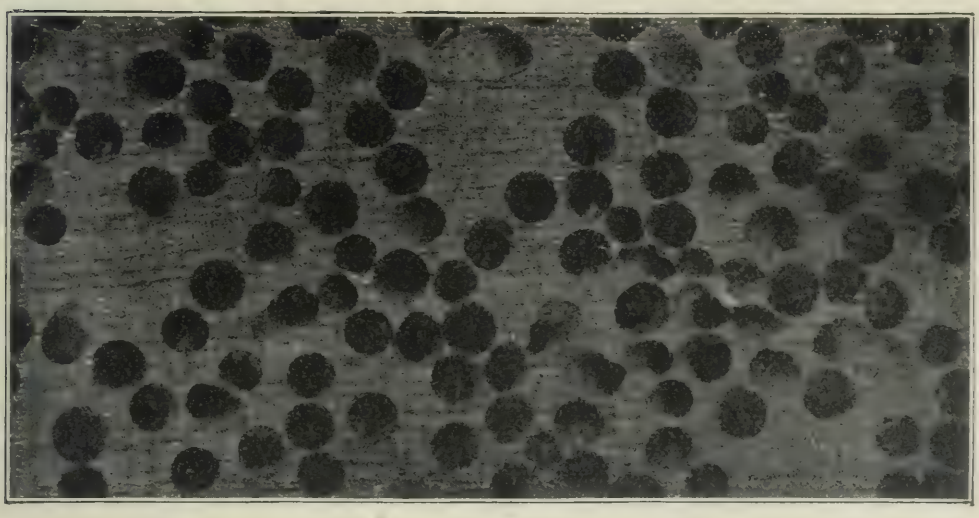

FIG. 2.—SECTION PARALLEL TO FACE OF FIG. 1. LIFE SIZE.

FIG. 3.—VERTICAL SECTION THROUGH FIGS. 1 AND 2. LIFE SIZE.

The Character of the Excavation.—The teredo is very small when it begins to attack the wood, and the hole by which entrance is made, which is the only perforation that appears upon the exterior, is very minute (see Fig. 1, Plate XVI). The animal develops very rapidly.

The adult diameter is usually attained within 1 or 2 ins. of the surface, and the burrow increases in diameter regularly from the point of entrance to the maximum diameter. The animal grows principally in the direction of length, and therefore, it attacks the wood so as to accommodate its quarters to this increase in length. The boring is first carried on across the grain, but ordinarily turns within a short distance and passes in the direction of the grain. This general direction is usually followed, but obstacles are so frequently encountered that the tunnels become exceedingly tortuous, and pass in every conceivable direction.

The teredo usually passes around knots, although quite competent to penetrate knots of oak and other hard woods. Adjoining tunnels are not encroached upon, because these tunnels are completely occupied by live teredos, and more ingenuity would be required to pass through one of them than to avoid it. When cracks exist in the centers of large timbers, they are approached from all sides, but the film is never willingly broken through. It may be assumed that the teredo does not desire to cross any crack or sign of cleavage. It prefers wood that is not surrounded by bark, because of the line of contact between the wood and the bark. When a piece of wood is thoroughly infested, the animals have to pass very close to one another, and the thin film of wood left between the adjacent tunnels is reinforced by the calcareous lining. E. L. Corthell, M. Am. Soc. C. E., states that the teredos pass through the willows of the Mississippi improvements so as to leave them a mass of nearly parallel tubes. The calcareous lining appears to lend some degree of strength and toughness to the wood thus weakened.

A fact which is of great importance is that the teredo must always command the opening of its burrow, and have free and permanent access to the water. One end of the teredo being thus fastened at the outside of the wood, the depth of penetration must be limited to the length of the animal. It cannot exist in the interior of woodwork, nor can it live or breed without actual contact with free water. A report that a teredo had been found in the interior of one of the

caissons of the Brooklyn Bridge was the cause of much popular and unwarranted excitement at the time of the erection of that structure.

It is impossible to examine the work of the teredo without surprise at the smooth surface of the burrow, which is cut as perfectly as by the sharpest chisel. In still weather the teredo may be plainly heard while at work. More than 50% of the weight of the wood may be removed by the teredo, without being greatly evidenced upon the surface. The little holes by which the animal gains entrance become partially obliterated when it dies. Wood may appear to be quite sound and yet be so weakened that much of it can be crushed by the hand (see Plate XVII). Failure, therefore, frequently comes suddenly. The tops of piles which appear to be in good condition are suddenly seen floating away. A large wharf at Provincetown, Cape Cod, unexpectedly collapsed by reason of the teredo. A freight train on the Louisville and Nashville Railroad crushed through a trestle which had been standing but ten months, and had been constantly inspected without showing signs of weakness. An examination showed that the piles in this instance had been eaten off close to the mud line.

Plate XVII is from a photograph of a log of Panama mahogany, which was cut in the uplands of the Isthmus and floated through fresh water to the harbor of Colon, where it remained floating in salt water while awaiting shipment. The log was overlooked for one season, and the work of the teredo is thought to have been accomplished in about nine months. The heavy, wet specimen was shipped with others, under the impression that it was sound, but it broke by its own weight after its arrival in New York.

The Size of the Teredo.—The size of the teredo depends upon the species, locality and age, and the absence of obstacles to excavation. Specimens of wood submitted for examination are frequently divested of any signs by which the excavators can be classified. The excavation can be identified as the work of the teredo, but the exact species accountable for the work cannot always be told.

Locality has much to do with development. Specimens grow more rapidly and attain larger size where the climate is warm. The teredo continues to grow until it reaches its maximum size, unless an obstacle is encountered. The species Navalis may be assumed to average from about one-fourth to three-eighths of an inch in diameter and from about 10 to 15 ins. in length, but specimens of the teredo frequently attain a

PANAMA MAHOGANY, DESTROYED IN ONE SEASON.

much greater size. Professor Siegerfoos writes that he has measured them up to 4 ft. in length and that the specimens thus measured had not arrived at their full limits. A log about 2½ ft. square, recently examined, was found to be entirely honeycombed, the borings starting from opposite sides and passing slantingly into the wood, so that they probably averaged over 2 ft. in length. The specimen was of particular interest because the destruction had been accomplished in a single season.

The minimum diameter and length of a boring may be taken as ¼ in. and 5 ins. respectively. The maximum length may be taken as 4 ft. The largest diameter ever noticed by the author measured 1⅛ ins.; this is shown on Plate XVIII. After the teredo has penetrated the wood for a little distance, the diameter remains about constant. Diameters are measured in this portion of the burrow and not at the entrance.

The Range or Field of Work.—The teredo operates throughout a vertical field of considerable depth. This field begins at a point a little above low-water mark, and extends downward until the pressure becomes too great, or the soil at the bottom is encountered. The teredo seems to be able to exist for a little time without submergence, and is therefore able to live above the low-water mark, although exposed daily between the tides. The interior extremity of the tunnel may be higher up than the entrance. The upper limit of the excavated wood cannot be determined by an examination of the orifices at the surface. The lower limit is uncertain and is probably different for different species. It has been assumed by many that the lower limit could be set at about 14 ft. below low water, but recent information, thought to be reliable, indicates that piles have been affected at a depth of from 20 to 25 ft. below that level. The fact that the interior extremity of the burrow is often found below the mud line has given the impression that the field of the animal may extend below this limit, but the outside opening or entrance made by the teredo is never below the soil, although the boring may turn downward for the whole length of the animal. If sediment accumulates around the bottom of the wood so as to cover the syphons, the death of the teredo results.

It is reported that in some harbors the teredos attack at the surface, and in others at the mud line. These differences are partially

due to differences in the constituents of the upper and lower layers of water. Where the fresh water of a river meets the heavy water of a sea, the teredo may be almost entirely confined to the lower stratum. The range or field of the teredo is important, because protective processes which could be confined to this field would be more economical than those in common use which are applied to the entire structure.

The Rapidity of the Work.—The rapidity of the work of the teredo depends upon conditions similar to those which govern its size. The evidence upon this subject is not always accompanied by a statement of the conditions under which the results were accomplished, such as the species of teredo, the character of the wood, the season, the climate and the depth of submergence, all of which are points as important as the geographical location of the work. The period in which the teredo accomplishes its work is variable. It may be six weeks or as many years, but rapid work is usually accomplished under the conditions which exist in warm climates.

Impure water and cold weather retard its activity, while pure or warm water expedites the work. Maximum probabilities being more important than minimum possibilities, it is safe to assume that a 6-in. boring may be driven in six weeks, and hence, as the animal attacks all sides, a pile 1 ft. thick may be destroyed in that period.

A young teredo has been found in wood which has been submerged for eight days[a]. Six-inch piles have been destroyed at Aransas Pass in six weeks[b], while other piles in the same locality have lasted for three or four months[c]. "Piles have been rendered useless by a submergence of one hundred days in Mobile Bay." On the Louisville and Nashville Railroad piles 12 x 15 ins. frequently have to be replaced after six months' service[d]. Unpainted spar buoys have a life of about one year in the vicinity of Cape Cod[e]. Piles have been destroyed in the harbor of Galveston in three years[f]. They have lasted twelve years in the Delaware Breakwater Harbor[g].

Reproduction and Development.—Mollusks produce their young by means of eggs. Those of the teredo are spherical in shape and

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[a] U. S. Annual Report of Scientific Discovery for 1857.
[b] Report, Chief of Engineers, U. S. A., 1888, pp. 13, 14.
[c] Annual Report, Chief of Engineers, U. S. A., 1879, p. 937.
[d] Montfort, *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 221.
[e] Report to U. S. Fish Commission, by Capt. Edwards.
[f] Report, Chief of Engineers, U. S. A., 1868, p. 512.
[g] Annual Report, Chief of Engineers, U. S. A., 1871, p. 667.

UNUSUALLY LARGE PERFORATIONS.

greenish yellow in color. The animal is exceedingly prolific: the eggs of a single specimen being probably numbered by the million. The eggs are first deposited in the gill cavity, and are almost at once fertilized. They are free-swimming at the end of three hours, have a well-developed shell before the end of the day, are very hardy, and all seem to be fertilized and to develop.

The embryo passes through several interesting stages before it assumes the character and form of the adult. It is first covered by fine hairs or cilia, which enable it to swim. These are soon lost, and the rudiments of a small bivalve shell appear, which is at first heart-shaped and very small, yet large enough to enclose the entire animal. The portion of the body which protrudes from the shell is fringed with cilia. These, again, constitute swimming organs, and the teredo swims actively until a piece of wood is encountered. The shell has now become rounder, and organs of sight and hearing have been developed. The appearance of these organs marks a climax in the life of the young animal, and it begins to elongate. The locomotive cilia disappear, the eyes are lost, and the mature form is gradually assumed. The life of the larvæ is about four weeks, during all of which time they are free swimmers. If the animal has become attached to wood, however, its energies may be expended thereon. The life of a specimen which has not found any wood to attack has not been determined, but is probably quite short.

The results of some observations by Professor Siegerfoos upon the Xylotrya Fimbriata at Beaufort are thus summarized:*

"The free-swimming stage is reached in three hours, and a well-developed shell is formed in a day. We have no direct observations as to the time the ship larva is free-swimming. We may assume, I think, that it is at least a month, or it may be two. Most of its energies are devoted to locomotion during this period, but, after it has attached itself, all of its energies are devoted to forming its burrow and securing its food. Coming in contact with the wood, the larva throws out a single, long byssus thread for attachment and never again leaves its place. The newly attached larva is somewhat less than .25 mm. long. In twelve days, it has attained a length of 3 mm. In sixteen days, 6 mm. In twenty days, 11 mm. In thirty days, 63 mm. In thirty-six days, about 100 mm., when it bears ripe eggs or sperm."

The extreme life limit of the teredo is unknown, but it is thought that under favorable conditions the animal may live for several years.

* Johns Hopkins Circular, 1896.

In the vicinity of New York the processes of reproduction take place for the most part in May. They are not entirely confined to that month, however, but may extend throughout a greater part of the summer. Reproduction in tropical countries is probably extended throughout the entire year. The animal may develop to a very large size, and may possibly attain maturity in a single season.

The Effect of Climate, Temperature or Water.—The Teredo Navalis thrives best under the influence of heat, but, notwithstanding this fact, it can resist cold to a considerable degree. It is not active when subjected to low temperatures, yet it can endure them. Some species of the teredo have been reported as far north as Eastport, Me., and they exist abundantly under such conditions as obtain at Cape Cod. Destruction is not carried on as continuously or as rapidly in cold climates as in warmer ones, and for this reason maximum results are seen along the South Atlantic and Gulf States and on the Pacific Coast, where the conditions are more favorable, and where reproduction is continued during a longer period.

The syphons of the specimens observed by the author contracted when the water became cold. Only the extremities could be seen when the temperature was about 45 degrees. The syphons expanded as the water became warmer, and were fully extended after several days of continued warm weather. The incurrent tube was stretched out in one instance to a distance of nearly 2 ins. The photograph (Plate XIII) was taken at this period. It is supposed that the work of excavation is not as active when the syphons are withdrawn.

The purity of the water should be considered in connection with the work of the teredo. Some species inhabit pure sea water; some prefer brackish water; others abound in waters that are muddy, while others again live only in waters that are clear and pure.* The teredo is often present in certain waters, yet absent in others nearly adjacent. This is usually due to some difference in the water. The Xylotrya Fimbriata seems to be able to survive the brackish, impure water of the inner New York Harbor, while other species could not live there, though they are present in the nearby outer ocean. The teredo is very active on the North Pacific Coast, yet is absent near the mouth of the Columbia, where the ocean is influenced by the outflow from the river.

* Percival Wright describes a kind of "ship worm" called *Nausitora Dunlopei* found in India, 70 miles from the sea, in perfectly fresh water.

An interesting incident is reported by a reputable firm in New York City. A vessel carrying hard-wood logs was wrecked in the vicinity of the Gulf of Mexico on a sandy beach separating the ocean from a river. The logs were thrown into the ocean, and were afterward beached and conveyed over the sand to the sheltered water of the river, where they remained about six weeks. The wood was rapidly affected as soon as it reached the brackish water of the river, the results being so noticeable that some of the borings were measured, and are said to have averaged 6 ins. in length. The wood which remained in the outer water was not injured.

It is stated that the Russians once built a large dock in the harbor of Sebastopol, and surrounded it with fresh water in the hope that it would be thus protected from the mollusks which infected the harbor; but it was found that the teredo destroyed the wood as rapidly as when it was submerged in salt water. The discrepancy indicated by these incidents may be accounted for by the difference in the species. The teredo which avoids the brackish water at the mouth of the Columbia differs from that which prefers it in the harbor of Sebastopol or in the river near the Gulf of Mexico.

The effect of the condition of the water upon the teredo is interesting. The opinion that the periods of unusual prevalence in Holland were in some way connected with a change in the quality of the water was expressed as early as 1733, and since that time has frequently been endorsed by Dutch engineers. Dr. von Baumhauer, Holland Commissioner to the Centennial Exposition has called attention to the fact that but little rain fell in the years when the teredo was so unusually prevalent, hence the smaller volumes of river water were thought to have permitted larger proportions of salt to reach the coast. This theory is strengthened by the fact that analyses showed a variation in the proportion of salt during dry and rainy seasons.

The Distribution of the Teredo.—The Teredo Navalis has been identified as existing in the United States between Florida and Cape Cod, and in Europe, from Sweden to Sicily. The *Teredo Norvegica* has been found from Cape Cod northward to the coast of Maine. The *Teredo Megotara* has been found in floating pine wood at Newport, R. I., and in cedar buoys, etc., at New Bedford, Mass. It has been found south as far as the coast of South Carolina. The *Teredo Dilatata* occurs from Massachusetts Bay to South Carolina. The *Teredo Thompsoni* has

been found at Cape Cod, Mass. The *Xylophaga Dorsalis* inhabits the waters of the North Atlantic. The *Xylotrya Fimbriata* is found along the Atlantic Coast from Long Island Sound to Florida. It also abounds in the waters of the North Pacific, and is one of the European forms.

Different species of the teredo are notably present in such localities as the Bermudas, Jamaica, New Zealand and Australia. The teredo, as a rule, may be generally found in the Tropics, and is hardly less numerous in many of the northern waters.

Woods Affected by the Teredo.—All varieties of wood commonly used in construction are subject to attack when exposed to the teredo. Immunity is occasionally claimed for some particular wood, but it will generally be found that the claims have been based upon local conditions and are not fully substantiated.

The only woods about which any doubt may be felt are those which contain some gum or bitter essence and those which have a porous structure ; and of these the majority are not well known to American constructors. Some of them undoubtedly possess merit as far as they have been tested, but others may be regarded as open to doubt, and it is an error on the safe side to assume that none are exempt under all conditions. A wood about which less doubt may be felt than any others is the Australian Jarrah. This is a variety of Eucalyptus which is much relied upon in Australia, but it is understood that it has failed in Ceylon, New Zealand and elsewhere. It is very possible, however, that the wood used at the works in question was not the Jarrah.

Karri wood is another form of Eucalyptus, the history of which is much less certain than that of the Jarrah, with which it is often confused. Both varieties are now being extensively introduced into Europe for street-paving purposes. It has often been held that Teak wood is exempt, but the evidence is against it, as Teak wood logs affected by the teredo have been received in New York City within a comparatively recent time.

The following list of partially exempt woods has been compiled by Mr. T. A. Britton from authorities which are said to be reliable:* (Western Australia) Jarrah, Beefwood and Tooart; (Bahama) Stopperwood; (Brazil) Sicupira, Greenheart; (India) Malabar Teak., Sisso,

* Treatise, "Dry Rot in Timber," p. 223

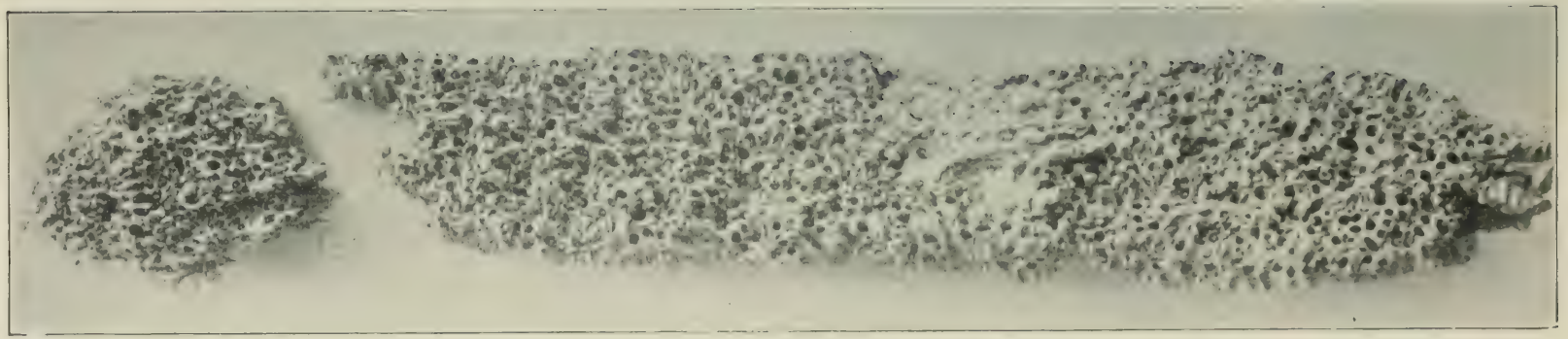


FIG. 1.—SURFACE OF WOOD ATTACKED BY LIMNORIA. LIFE SIZE.

FIG. 2.—LIMNORIA FROM SPECIMEN SHOWN IN FIG. 1. LIFE SIZE.

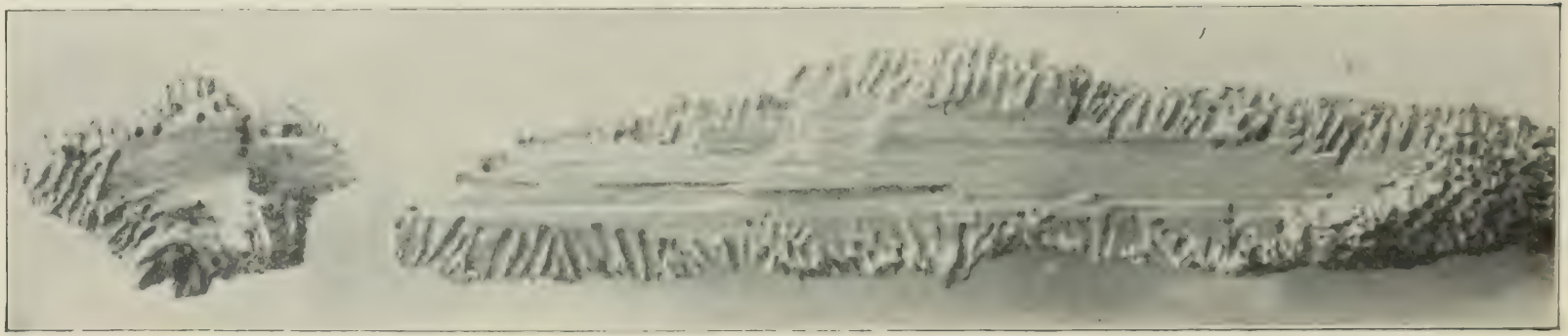


FIG. 3.—SHOWING DEPTH OF HOLES. LIFE SIZE.

May-Tobek; (South America) Santa Maria Wood; (Tasmania) Blue Gum; and (West Indies) Lignum Vitæ.

It is not urged that these are entirely exempt, but that they have been exempt for long periods. Very few of them are widely known in construction. It is understood that at Southampton some greenheart piles have failed recently.

A commission, appointed in Holland to investigate this question, decided that—

"Although we do not know with any certainty if among the exotic woods there may not be found those which resist the teredo, we can affirm that hardness is not an obstacle that prevents that mollusk from perforating his galleries."

This conclusion is well borne out by the experience with the Iron Bark tree, the Eucalyptus Lencoxylon, of Australia. This wood has a very great tensile strength, and the crushing strength is said to be "nearly one-fourth that of iron." The wood is certainly very hard, and yet the teredo is by no means repelled by it.

The Eucalyptus Globulus, or Blue Gum, has been successfully introduced into the United States, and is grown in California and Florida. Some hopes have been expressed that this wood might be useful for marine purposes, but they have not been realized. Cedar has repelled the teredo for some little time in the harbor of San Francisco, and it is stated that some species of the Black Mangrove in Jamaica are exempt.

The Osage Orange, or Bodark, has been used to some extent in the Gulf States. It is understood that this wood is relied upon principally because it is hard.

A much greater reliance may be placed upon structure than upon the presence of foreign substances. The teredo desires a compact wood for its abode, and does not like cracks or loose structure. Endogenous trees, palms, for instance, are probably exempt to a greater or less extent. The wood of the latter consists of a mass of thick fibers so independent of one another that brushes can be made by rubbing one end of a stick until the fibers become detached and appear like bristles. Considerable reliance may be placed upon the Cocoa Palm of Mexico, the King Palm of Cuba, and the Cabbage Palm or Palmetto of Florida.

It may be assumed that the conditions of impregnation or structure necessary to repel the teredo do not exist naturally in such woods as

are commonly used in engineering works. It may also be assumed, so far as known at present, that partial or complete immunity, as applied to such woods as are in common use, is a question of locality rather than of variety of wood.

The Limnoria Lignorum, White.

This small crustacean has several names, as the Limnoria Terebrans, the Gribble and the Boring Gribble. The limnoria has not been studied for so long a period as the teredo. It was first noticed by Robert Stevenson in 1810, and was examined by Dr. Leach, who one year later pronounced it a new species. It has been investigated since that time by numerous European writers, and, in the United States, it has been studied by Dr. Verrill, of Yale University, and Dr. Sidney I. Smith, of the United States Fish Commission.

The limnoria is gregarious and is found, if at all, in large quantities. It is much smaller than the teredo, but it exists in greater numbers. It has been traced from New York northward to the Bay of Fundy, and large numbers exist in the North Pacific Ocean. It is a very familiar and destructive form of life in Europe. If the destruction accomplished by the limnoria could be estimated it would be found to be surprisingly great.

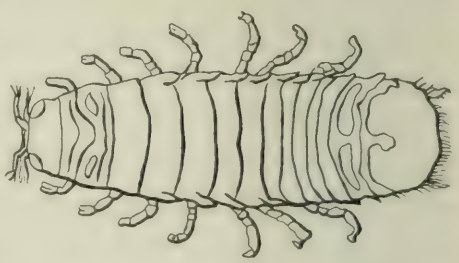
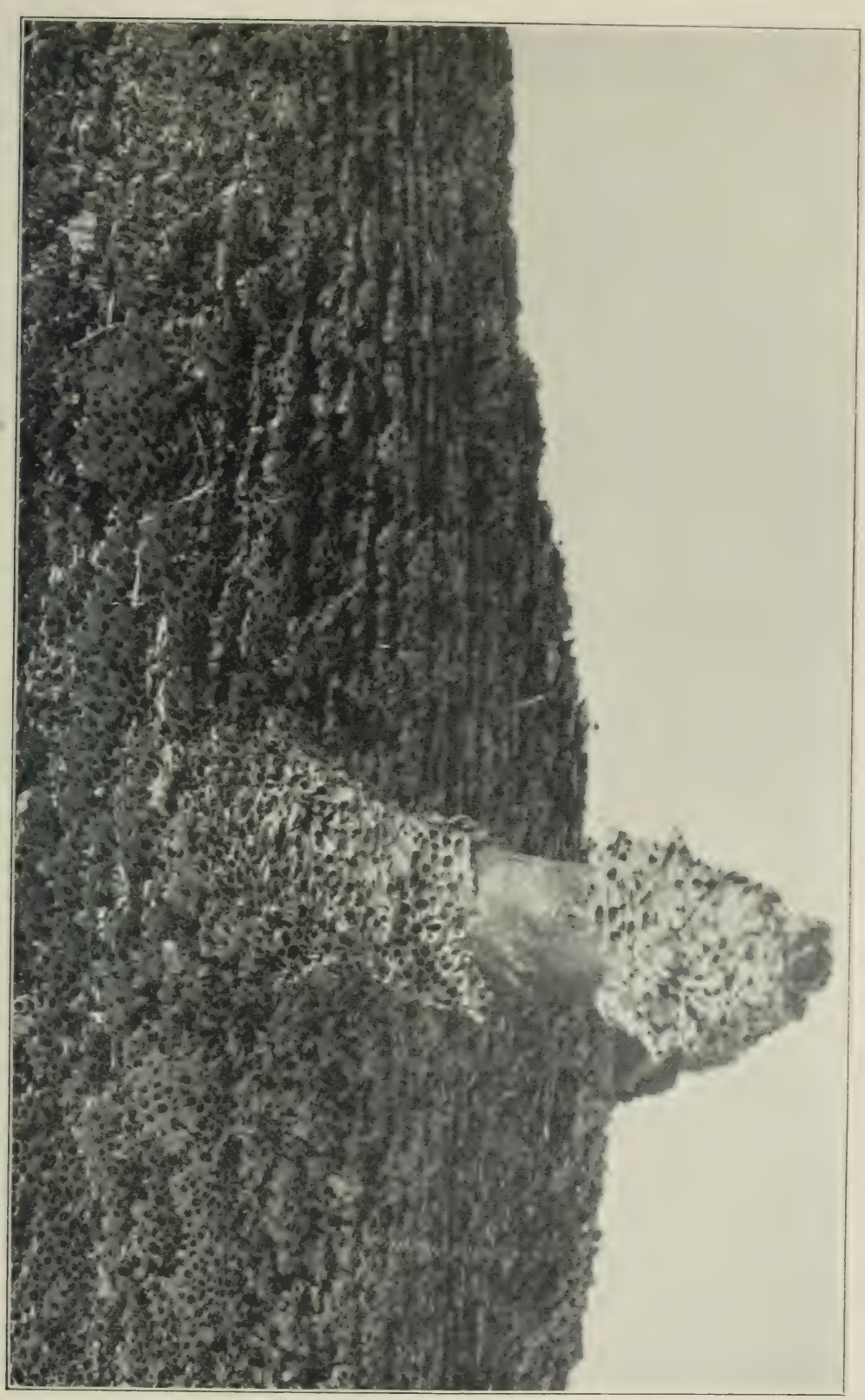


Fig. 3.

Descriptive.—The limnoria (Fig. 3) is about as large as a grain of rice. The body is flat, round at each end, and consists of fourteen segments. The sides are nearly straight and are parallel to one another. To each of the seven segments which follow the head is attached a pair of short, stout legs terminating in claws, the shape of which suggests the small claws of the lobster. The upper surface of the body is covered with small hairs to which more or less dirt usually adheres. The body is grayish in color, and sometimes resembles the color of the wet wood so much that it is difficult to distinguish it. The limnoria can swim, creep backward and forward, as well as jump backward by means of its tail. When touched, it rolls itself into a ball, and in this particular, as well as in general appearance, it resembles the common sow bug.



PILE FROM PUGET SOUND, SHOWING WORK OF LIMNORIA. LIFE SIZE.

Vital Processes.—The limnoria differs from the teredo in that it is a vegetarian. The teredo is sustained by infusorial life, but the limnoria devours wood. Its tunnel affords both food and shelter.

Boring Apparatus.—The limnoria attacks the wood by means of its mandibles or claws. It prefers wet wood and succeeds in making a very clean-cut excavation.

Character of the Excavation.—The work of the limnoria differs from that of the teredo in that it works upon the surface of the wood in such a manner as to be clearly seen, while the work of the teredo is usually concealed until the failure of the wood. The limnoria is similar to the teredo in that its tunnel must communicate directly with the salt water; hence neither of these animals can live in the interior of thick woodwork, such as that of a caisson. The limnoria makes a small, round, parallel-sided tunnel through which it can pass freely back and forth from the sea. The diameter of the entrance of the tunnel is similar to the average diameter. The tunnels are quite short, and are placed very close together (see Plate XIX). They are so numerous that the wood is rapidly reduced to a series of very thin partitions, which soon decay or are washed away by the waves, thus exposing a fresh surface which is at once attacked. Layer after layer is thus rapidly removed, so that the timber is destroyed in a very few years. The limnoria frequently works in conjunction with the teredo, attacking the exterior while the teredo destroys the interior of the wood, and this combination effects a rapid destruction.

The limnoria attacks both the hard and soft parts of the wood. The hard annual layers have not been avoided in the specimens examined. The limnoria can penetrate knots, but frequently avoids them, so that these hard portions stand out in relief as the timbers waste away. Iron rust is said to cause a somewhat similar effect.

The Size of the Limnoria.—The limnoria is very small, but notwithstanding this fact, it is very destructive. The multitude of these animals compensates for their size. It may be assumed to be from $\frac{1}{6}$ to $\frac{1}{4}$ in. in length, and about $\frac{1}{16}$ in. in diameter. The tunnels are about $\frac{1}{2}$ in. in depth.

The Range or Field of Work.—The wide range observed between the several species of the teredo does not apply to the limnoria. Its work, as observed in the United States, is generally confined to a limited distance above and below the low-water mark. Where the varia-

tions of the tides are extensive, as in the vicinity of the Bay of Fundy, the range of the limnoria is correspondingly great. The United States Fish Commission states that it has been found, although rarely, as deep as 40 to 60 ft.

The Rapidity of the Work.—The limnoria does not work as rapidly as the teredo. The number of individual workers may be taken as a measure of the work they accomplish. The number of tunnels is more important than their depth. Limnoria are almost invariably found in large numbers and destroy a layer from $\frac{1}{4}$ in. to 1 in. in thickness in a year, the average yearly destruction being probably $\frac{1}{2}$ in.

Almost all wood used in marine locations is in the form of piles, which are necessarily exposed upon all sides. Their effective diameter may be reduced at the rate of 1 in. for each season, which result, while not equal to that accomplished by the teredo, is sufficient to cause a great loss.

The Effect of Climate, Temperature and Water.—The limnoria is found where the coldness of the climate prohibits the existence of the teredo. It requires pure sea water, and cannot exist in fresh or in impure water, consequently it is not found at the mouths of rivers.

The Distribution of the Limnoria.—The animals are distributed along the American coast from Florida to Nova Scotia. They exist sparingly in Long Island Sound, but are quite numerous upon the coast of Massachusetts, and are very destructive in the Bay of Fundy. They are very active along the North Pacific coast, and are much feared in the vicinity of Puget Sound and the Straits of Fuca. They exist also in abundance upon the coast of Great Britain and in other parts of Europe.

Woods Affected by the Limnoria.—The limnoria seems willing to attack all varieties of wood commonly used by American constructors, but is said to prefer soft woods. It has been known to attack the gutta percha of submarine telegraph cables. It is said that teak wood is free from attack. Plate XX is a life-size photograph of part of a piece of wood from Port Townsend, Wash., showing the work of the limnoria.

Sphæroma Destructor (Richardson).

Attention has recently been called to this hitherto undescribed form of life. This animal is interesting in that it is active in comparatively fresh water. It resembles the limnoria in that it attacks the wood

FIG. 1.—WORK OF DESTRUCTOR.

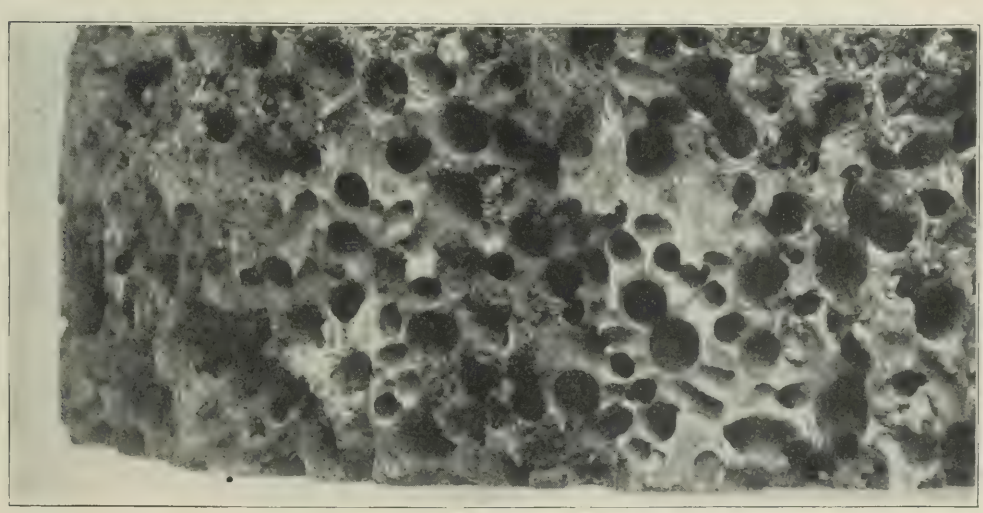


FIG. 2.—SURFACE VIEW OF FIG. 1. LIFE SIZE.

from without, the interior of the wood being unaffected while the exterior is being destroyed.

The work of these animals was first noticed upon some of the trestles of the Florida East Coast Railway, in the vicinity of St. Johns River, in Putnam County, Florida. Specimens of the wood were submitted to the Carbolineum Wood Preserving Company, of New York City, and were referred by them to the Smithsonian Institution at Washington, where they were studied by Miss Harriet Richardson.*

The animal somewhat resembles the limnoria in appearance, and is dark brown in color. It works between high and low-water marks. These are not tidal levels, but changes due to the wind assisted by the tides. The water appears to be quite fresh and the water hyacinth, which is not commonly found in salt water, flourishes in the vicinity. The distance to the ocean is about 100 miles.

The diameter of the long-leafed yellow pine pile, from a photograph of which Plate XXI was prepared, is said to have been reduced from 16 ins. to $7\frac{1}{8}$ ins. in eight years.

Several forms of fresh-water shipworms (some very large) have been found in Australian rivers.†

The Chelura Terebrans.

This animal was first noticed at Trieste in 1839, and was next found in some piles in the harbor of Kingston. The Irish specimens were described by Professor Allman in 1847.‡ The chelura was not identified in America until 1875, when two small specimens were discovered by Professor Sidney I. Smith at Wood's Holl, Mass. No others were observed until August, 1879, when Professor Verrill discovered a number of them in some piles at Provincetown, Mass. The chelura unquestionably belongs to the amphipods, and there is apparently but one species of the genus. The C. Pontiac described by Czerniavski in 1868 is identical with the Chelura Terebrans.

Descriptive.—The general appearance of the chelura (Fig. 4) resembles that of the ordinary shrimp, and for this reason is sometimes referred to as the wood shrimp. Its shape differs from that of the limnoria in a very striking degree. The two animals resemble one another only in size. The chelura is a very active little animal, and

* Paper before the Biological Society of Washington, D. C., May 13th, 1897.
† Charles Hedley, F. L. S., Sidney, Australia.
‡ Ann. and Mag. Nat. Hist., xix, 1847, p. 361.

swims upon its back. It is a jumper, and can project itself to a considerable height when placed upon dry land, and in this respect resembles the sand hopper. The body is semi-translucent, and is thickly spotted or mottled with pink. The animal is distinguished by three pairs of caudal stylets, the last of which are nearly as long as the body. Those of the females or the young animals are not so long.

Vital Processes.—The chelura resembles the limnoria in that it is a vegetarian, and its burrow affords both residence and food. The fact that the chelura devours wood for sustenance is proved by the minutely divided ligneous matter found in the alimentary organs of dissected animals.

Boring Apparatus.—Professor Allman's original study of the chelura is in part yet regarded as authoritative. He states that the chelura attacks the wood and reduces it to minute fragments by means of a kind of file.

The Character of the Excavation.—Great difficulty has been experienced in obtaining specimens of the work of the chelura, and those obtained are not sufficient to warrant many generalizations. In many particulars the work of the limnoria and of the chelura bear such a close resemblance as to lead to the suspicion that these animals are sometimes confused with one another. The excavations of the chelura are slightly larger than those of the limnoria, but are conducted in much the same manner, as the wood is attacked entirely from without. Numerous punctures are made, and then the weakened layer succumbs to the action of the waves, the surface thus exposed being in turn attacked and the wood destroyed in the same manner. It is stated that the excavations of the chelura are more oblique in their direction than those of the limnoria, and this is certainly true of the specimens observed.

FIG. 4.

The chelura appears to prefer soft wood, and their attacks are made as much as possible in the softer annual rings. The tendency toward an arrangement of perforations in lines is shown in Plate XXII. The work of the chelura differs from that of the limnoria, in that the latter attacks the wood at any available point, while the chelura, on the contrary, prefers the softer portions, and avoids the hard wood

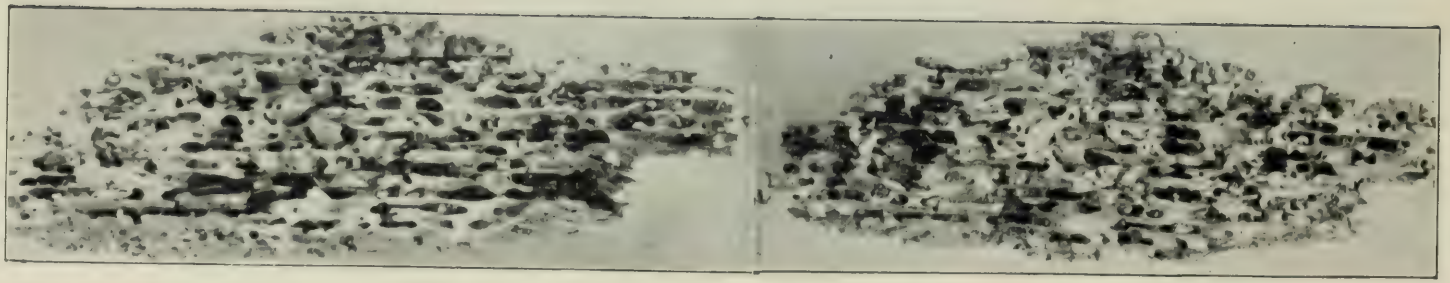


FIG. 1.—SURFACE APPEARANCE OF WOOD ATTACKED BY THE CHELURA. LIFE SIZE.

FIG. 2. LIMNORIA FROM SPECIMEN SHOWN IN FIG. 1. LIFE SIZE.

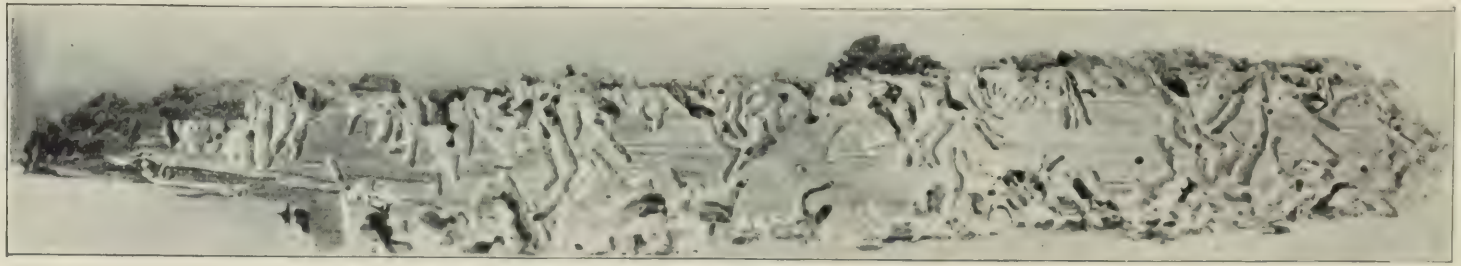


FIG. 3.—SECTION OF SPECIMEN SHOWN IN FIG. 1. LIFE SIZE.

around knots. Perforations found in such localities may be assumed to be the work of the limnoria.

The chelura and limnoria are associated with one another in the American localities which have afforded specimens of the former. The perforated wood from these localities shows that the limnoria were in the timber in advance of the chelura. It also shows larger individual chelura confined to the soft rings. These facts indicate that young chelura may at first follow limnoria through the hard rings, but that as they increase in size, they turn toward the soft wood. They obviously attain full size in the larger tunnels. The individual chelura appears to be even more formidable than the individual limnoria.

The Size of the Chelura.—The chelura is somewhat larger than the limnoria. It is said that specimens one-third of an inch in length have been measured.

The Range or Field of Work.—The frequent confusion between these two animals, together with the lack of American data, leaves the question of range unsettled. The specimens found at Provincetown were all taken from wood submerged from 8 to 12 ft. below low-water level.

The Distribution of the Chelura.—The chelura was sought many times along the American coast between New Jersey and Nova Scotia, but was not discovered until 1875. It is yet confined, so far as known, to the two original localities, Wood's Holl and Provincetown, both in Massachusetts, but it is more than possible that the animal has escaped observation, and that it is common on the North Atlantic coast. The unskilful eye would readily confound the chelura with the limnoria, although the two animals belong to distinct divisions of the crustaceans. It is quite possible that some of the damage hitherto ascribed to other animals has been accomplished by the chelura.

The chelura has been reported at many places on the coast of Europe, and is mentioned as existing from South Norway to the Adriatic. Attention has been called to the extent of its range. It is said to be an inhabitant of Australia.* In Europe a very great amount of destruction is attributed to this species, and efforts have been

* Since the above was written Mr. Charles Hedley has ascertained "that not only are Chelura and Limnoria not included as Australian pests, but that neither are known to exist in Australian waters. We have, however, kindred forms which take their place and do their work."

made to substantiate these points, but have thus far been unsuccessful. It may be that some European results, attributed to this animal, are deserved by the limnoria, as it is probable that some of the work of the limnoria in America should be attributed to the chelura, and it is more than probable that the animals are frequently associated. Efforts to discover particular works affected exclusively by this form of life in Europe have not thus far met with success. The chelura has earned a most unenviable reputation in Europe, but it is not known in which places it exists as a specimen and in which as a pest.

Sphæroma Vastator.*

This species was found in the woodwork of a railway bridge on the west coast of the Indian Peninsula. It is said that it honeycombs the wood with cylindrical holes.

The Pholos.

A description of the pholos may suitably accompany any mention of marine wood-borers. This animal does not attack wood, but penetrates the hardest stone, and is therefore interesting because it illustrates the power of boring animals.

The pholos and the teredo are nearly related. The former differs from the latter in that its shell is much larger and in form more closely resembles the long clam. Some species of this animal are much prized by the French as table luxuries. Others are used as food on the North Pacific Coast. The pholos is an inhabitant of many seas; it is plentiful in the English Channel, and is found in many places on the American coast. The borings of the pholos are instructive, from a geological point of view. The so-called Temple of Seraphis, near Naples, affords a prominent illustration of the movement of the earth's crust. This temple has sunk in the water and has then arisen again, the fact of submergence being made clear by the perforations of the pholos. The three principal columns are honeycombed up to a height of about 10 ft., which shows that the ocean once covered the columns to that height.

The method of excavation has already been described in comparison with that of the teredo. The pholos opens its shell so as to brace itself

* Described by Mr. Spence Bates in the Ann. of Nat. His., Vol. xvii, 1886, pp. 28-31.

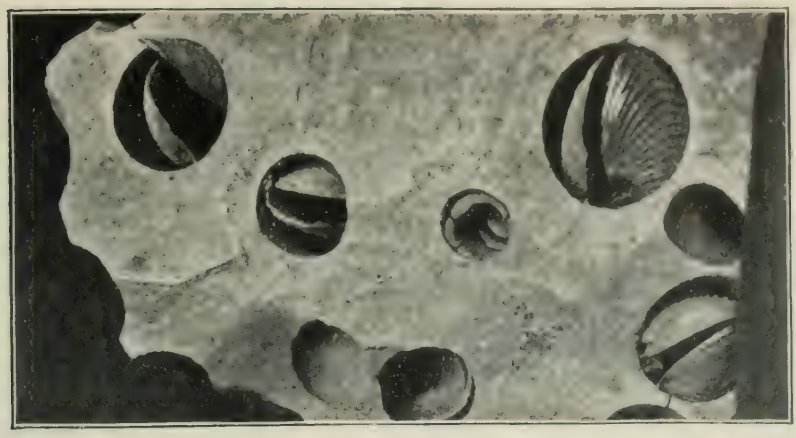


FIG. 1.—PHOLOS, EXCAVATING SANDSTONE. LIFE SIZE.

FIG. 2.—DACTYLUS, EXCAVATING GRANITE. LIFE SIZE.

FIG. 3.—MARTESIA XYLOPHAGA, IN PANAMA MAHOGANY.—LIFE SIZE.

against the sides of its tunnel. The long foot or pestle, which is similar to that of the teredo, emerges and rubs at the surface of the stone. It is assisted by the particles of sand or rock. The cavity is thus enlarged to accommodate the growing animal. Fig. 1, Plate XXIII, shows one of the numerous species of this animal at work upon a piece of sand-stone. Fig. 2, Plate XXIII, shows a similar animal perforating solid granite; while Fig. 3, Plate XXIII, exhibits another form, found in hard wood.

The series of marine stone-borers is very great, and includes the numerous species of the pholos family, together with other animals not related to them. One of the animals of this class is a powerful enemy of the oyster industry. Another of them destroyed in one year a cargo of marble which had been wrecked in the North Atlantic.

Barnacles.

The barnacle does not perforate wood, but usually attaches itself singly or in clusters to floating or submerged wood, and does not injure it. It is removed from the bottoms of ships because it impedes their progress. Barnacles protect the surfaces they cover. The white blotches on Fig. 1, Plate XVI, show the places where barnacles were at one time attached. Fig. 3, Plate XIV, shows the form of the barnacle.

Methods of Protection.

A history of the attempts to preserve wood from the attacks of wood-borers would be voluminous. It is only necessary to call attention to those methods which have been attended with more or less success. Most of the attempts in this direction have been made with the idea of protecting wood from the attacks of the teredo. It happens, fortunately, that any method insuring immunity from the teredo secures wood from other wood-borers as well. The methods which have been used may be classified as follows:

Removal During the Breeding Season.—This method may be used to protect such objects as buoys, bathing-houses and row boats. It is only applicable where the breeding season is short as it is in the North.

A Change of Water.—Wooden vessels which have been attacked by the teredo are sometimes hauled into fresh or muddy water. The animals which have gained entrance to the wood are killed by this

means. The suggestion has been made that expensive wood-work subjected to the teredo be surrounded by fresh water.

The Use of Selected Woods.—The few varieties of wood for which claims in this particular have been made are not widely known or employed, and it is seldom urged that any are permanently exempt. Repeated attempts have been made to discover some wood upon which reliance could be placed, but with meager results. The evidence, thus far, is in favor of the palm and the Australian jarrah wood.

External Coatings.—Many of the protective methods may be grouped under the head of external coatings, one advantage of which is that the treatment may be limited to that portion of the wood which is exposed to attack, while those parts which are below the mud line or above high water need not be considered. This is not the case where internal treatment is used.

(*a*) The bark is sometimes left upon the wood, and, as long as it remains intact, protects it from the teredo. This is explained by the reluctance of the teredo to cross seams. The bark is soon loosened and removed by the waves, however, and the wood is then exposed. It is doubtful whether bark serves as a protection against the limnoria and chelura.

(*b*) Thin plank, joined closely upon the surface of the wood, will preserve it from the teredo during the existence of the plank, but affords no protection from limnoria or chelura, for the plank is soon honeycombed or loosened and the interior woodwork is then exposed to attack.

(*c*) Metallic sheathings, such as copper and zinc, have been successfully used in many places, and the former has proved to be one of the most valuable methods of protection when placed upon piles so that when they are driven home the metal extends below the mud line and up to or above high-water mark. It is used upon the bottoms of wooden vessels and is much superior to zinc, which is quickly acted upon by the salt water. Metallic coatings are expensive, but are very effective, however, in protecting against all forms of marine borers. Surfaces sometimes become coated with barnacles and similar animals which afford further protection to the wood.

(*d*) Teredo or "worm" nails have been extensively used,* and are said to have originated with the Romans. They have short spikes

*The Dutch use them to a height of one-half tide.

and large, flat and sometimes square heads. They resemble ordinary carpet or upholsterers' tacks, and are driven close together. According to the specifications adopted by the Dutch Waterstaat, the nails must be well forged and not brittle. The diameter of the head must be 3 cm. and the length of the tack 4 cm. One kgr. is to contain from thirty to thirty-four nails. An interesting series of experiments with teredo nails has been conducted by the New York Department of Docks.* Teredo nails are apt to rust and thus cause failure. It was once held that the iron rust impregnation assisted in repelling the teredo, but this appears to have more foundation of truth as regards the limnoria.

(*e*) Paints, verdigris, paraffine, tar, asphalt and other mixtures have been used as protectives, but it is usually difficult to retain such coatings in position. Mixtures which withstand the softening effect of sea water are likely to be removed by erosion, and surfaces should be inspected at least once a year.

(*f*) Attempts have been made to combine paint mixtures with some fabric such as burlap or wire netting. Asphalt and net have not proved successful on the Pacific coast,† but a combination of paraffine paint and burlap used there has attracted attention. After removing the bark the surface of the pile is covered with a prepared compound, some of the ingredients of which are paraffine, powdered limestone and kaolin. The pile is then wrapped in jute burlap, and another application of the compound is made. Wooden battens are then nailed along the surface, which receives a final coat of the paint. Piles thus protected have been in use for ten years on the Pacific Coast by the California State Board of Harbor Commissioners, by the Northern Pacific, Great Northern and other railways, and are said to have been successful. The coating protects the piles from the teredo, limnoria and similar animals, but its duration is not known.†

(*g*) Piles are sometimes covered with Portland cement mortar. The bark is first removed and the wood cleared of knots and similar projections. The pile is then driven to its final position. The mortar is applied in several ways. A jacket of ordinary sewer pipes extending from the mud line to high-water mark is sometimes placed around the pile, and the space thus enclosed filled with hydraulic cement. Piles

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 235.

† *Engineering News*. February 8th, 1894.

thus protected have been put in position and observed by the California State Harbor Board. The coatings were soon found to have cracked, probably because they were too stiff. An iron shell or mould made in two pieces, bolted together tightly around the pile has sometimes been used. The intervening space is filled with cement, and the mould removed as soon as the cement has become hard. The Louisville and Nashville Railroad treated four thousand piles in this way, at an average cost of $1.25 per foot of length.* The cost of repairs for the first seven years was comparatively small. The concrete became coated with oysters and barnacles and was thus further strengthened. The advantage of such a treatment is that it can be applied after the piles have been driven. Teredos or limnoria may unexpectedly attack the wood, and any specimens which have gained entrance can be killed and others repelled by this method. The cost is not as great as might appear, since the entire length of the pile is not covered.

(*h*) The use of sand has been found to be both effective and low priced. Cylinders of earthenware pipes joined together by a special cement, are lowered over the pile and pushed into the bottom. The space between the cylinder and the pile is filled with sand. Any fracture or leakage is made evident at the top and can at once be made good. This method was suggested by the Louisville and Nashville Railroad, and is considered to be an improvement on their former method of protection by means of cement, while the cost, about 70 cents per foot, is much lower. The method is said to insure greater elasticity and better protection at the bottom. Piles treated in this way on the New Orleans and Mobile Division are apparently as sound as when driven, twenty years ago. In some cases their tops were not covered with pitch when they were sawed off, and the heart wood of a few of these specimens has decayed. The outer sapwood still remains sound.†

(*i*) External protection is sometimes afforded naturally. The surface of the wood may become covered with barnacles, mussels, oysters or similar animals, and is thereby protected from attack. Sea thorns sometimes multiply to such an extent that the entire surface is covered by their disks, which afford a very effective protection.

A Dutch commission, after six consecutive years of investigation, reported that coatings applied to the surface of wood seemed insuffi-

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 225.

† *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 221.

cient; that such coatings are likely to be injured by mechanical means; that chemical changes are to be looked for; and that it is difficult to obtain a covering which will continue in close contact with the wood.

The subject of external coatings may be thus summarized: Protection may be afforded as long as the coating remains intact, but this is difficult to accomplish.

Internal Treatment.—Many substances, such as water-glass, the salts of mercury and of iron, have been suggested as substitutes for coal-tar creosote, but none of these can compare with it, and therefore deserve no further notice.

Creosote supplies the best means for repelling the attacks of the teredo, limnoria and other sea animals, and also the termite and other land wood-borers. The subject of creosoting divides itself into three parts: the creosote, the method and the wood.

Creosote is a substance which is contained in the second distillation of coal tar. The first distillation consists of light oils, the second, creosote, and the third, pitch. Tars differ greatly in their chemical constituents, and in their products of distillation. The word cresote, therefore, has not an absolutely exact definition. The substance has no chemical symbol, as it applies to a fluid, the constituents of which constantly differ. It is essential that creosote should be heavier than water, as light creosotes have never been satisfactory, and most of the failures attributed to creosote have really been due to the use of such oils.

Creosote is expected to act in two ways. It introduces antiseptics into the wood; it also fills the pores with thick, gummy insoluble oils and naphthaline. Therefore, a second distillate of coal tar, which contains antiseptics and gummy substances in sufficient quantity and of satisfactory quality, should be selected. It should contain over 40% of naphthaline, and as little pitch as possible. It may contain as much carbolic acid as is likely to be present in this distillate, which will not be over 4 or 5 per cent. No substances likely to accompany the minimum of 40% of naphthaline will be injurious, because many of them may be regarded simply as vehicles. Heavy oil of creosote is heavier than water, and is sufficiently insoluble to remain in the wood for a long time. Creosote weighs from 8 to 9 lbs. to the gallon. The United States cannot meet the demand for dead oil of coal tar, and,

therefore, a large quantity is derived from England. The so-called "London Oil" is very thick and heavy. It is thought to be one of the best grades of creosote for marine work.

The method by which creosote is introduced into the wood is most important, but any method which will insure a thorough impregnation will be satisfactory. The wood is first heated in a vacuum to remove the moisture. The heat is so manipulated as to vaporize the sap and coagulate the albumens of the wood. Heated creosote is then introduced, and the condensation of the vapor in the wood causes a vacuum which, assisted by pressure, draws in the creosote. A gauge outside of the tank indicates the subsidence of the creosote as it passes into the wood. The process is stopped as soon as the specified quantity of creosote, usually from 10 to 16 lbs. per cubic foot of wood, has been forced in.

The selection of the wood which is to receive the creosote is important. Some woods are more porous than others, and one which will permit the free entrance of creosote is better than one which is hard or otherwise durable. It should be of such a nature that it will protect the creosote after impregnation. Creosote has occasionally failed because it has not been used in connection with wood of the proper quality. The Georgia pine and the Loblolly pine are the best for this purpose. Green woods are sometimes preferred to those which have been seasoned, because the condensation of the vaporized sap assists in more thoroughly impregnating the wood.

Many cases of failure are recorded against creosoted wood. Other cases are on record of woods which have resisted at first, but have succumbed after several years of exposure. In all these cases it may be assumed that the creosote was at fault; that the work was not thoroughly done, or that the wood was of a kind which resisted the ordinary treatment. A well-selected wood, thoroughly impregnated with good coal tar creosote, will resist the teredo, the limnoria, and probably all other forms of life for many years.

Substitution.—Substitution can hardly be classified as a method of preservation, but should be mentioned in connection with this subject, because the use of iron in ship-building is constantly increasing. Were this not so, marine wood-borers would require much more attention than they receive at present. Iron piles are used to some extent, and the use of iron work in marine construction may be safely

said to be on the increase. There are many marine works, however, in which iron can hardly be used as a substitute for wood.

The author has received notable assistance from Professor Verrill, of Yale University; Professor Packard, of Brown University; Professor C. O. Siegerfoos, of Johns Hopkins University; the National Museum at Washington, and General John M. Wilson, M. Am. Soc. C. E., Chief of Engineers, U. S. A. The New York Museum of Natural History has permitted the author to photograph some of its specimens. The United States Fish Commissioner has contributed much data, and prepared drawings of the teredo, the limnoria, and the chelura. The New York Aquarium, Colonel William Ludlow, M. Am. Soc. C. E., Francis Collingwood, M. Am. Soc. C. E., T. G. Hoech, M. Am. Soc. C. E., and many others have been of assistance.

DISCUSSION.

Mr. Berg. WALTER G. BERG, M. Am. Soc. C. E.—Attention should be called to the beneficial effect of emptying sewers and factory refuse near timber structures. It is a well-known fact that, in New York City, piers or harbor timber structures located where the water is heavily impregnated with sewage or factory refuse are not seriously damaged by the teredo. This fact is almost conclusive proof that these animals require for their existence clear, good, salt water.

Mr. Harrod. B. M. HARROD, Past-President, Am. Soc. C. E.—The engineers of the Louisville and Nashville Railroad have had a very large experience, perhaps the largest in the country, in fighting this mollusk on the Gulf Coast. They have used creosoting extensively, and for this purpose have erected their own works. They have tanks 6 ft. in diameter and 100 ft. long. The sap is extracted by the condensation of the steam, and then the creosote is put in under pressure. Their bridge piles are now standing in good condition, after some 12 years. They have also used cement around the piles, and another method which they have tried seems to the speaker to be very attractive, but he does not know that it has yet been fairly tested. After driving a pile, drain tiles are slipped over it, thus forming a very loosely fitting pipe. The tiles are placed in position by a diver, and then simply filled in with sand. The speaker thinks this is the last experiment they have made and that it is giving good results. It is certainly the cheapest plan yet tried. In connection with this subject, it will be interesting to refer to the paper by R. Montfort, M. Am. Soc. C. E.*

Mr. Manley. HENRY MANLEY, M. Am. Soc. C. E.—In Boston harbor the teredo, as a rule, is not found. The Cape Cod peninsula seems to be the dividing line; but territory upon the border line may be subject to invasion, and two or three years ago there was a considerable inroad which was wholly unexpected and unprepared for. In this case a dredging company was operating at the mouth of the harbor. In the early spring the company brought some new hard-pine lumber from the South, which had been floated down one of the rivers of Georgia and shipped to Bath, Me. Two large scows, each holding perhaps 500 or 800 cu. yds., were built of this lumber and brought to Boston early in the spring. They were pulled out of the water to be measured by the Government engineers, and then used in the work at the mouth of the harbor. Late in the summer they began to leak unaccountably. On examination they were found to have been thoroughly riddled and bored by the teredos, which were good-sized specimens and of full growth. It was rather a shock to the people having the care of wooden structures in Boston harbor, and they were a little appre-

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 221.

hensive; but the following winter destroyed all the teredos, and since Mr. Manley.
that time they have not reappeared. In Boston it has not been thought necessary to use any protection against the teredo, and in an experience of 25 or 30 years the speaker has known of only six or seven isolated cases where the animal has appeared, and then, except as above stated, only as specimens.

The limnoria has been found in Boston harbor from time out of mind, and the local engineers find no particular difficulty in guarding against its attacks.

It has long been the practice to specify that in exposed situations the bark upon piles shall be intact, and that the trees used for this purpose shall have been cut at such seasons that the bark will retain its place. So long as the bark remains, the experience has been that it is a perfect protection from the limnoria.

B. L. Crosby, M. Am. Soc. C. E.—Some years ago, on a visit to Mr. Crosby.
the East, the speaker took a trip in Boston harbor to what was then called Downer's Landing, and, at low tide, the water being very clear, he noticed that a large number of piles were nearly eaten off; the type of destruction being that of limnoria. That is, the wood was gone, so that the points of many of the piles near the ground line looked as though they had been sharpened like a lead pencil. A new wharf was then being built and was only finished in time.

Robert Moore, M. Am. Soc. C. E.—A new process for preserving Mr. Moore.
piles consists of the application of a cement of a somewhat new type called lithocite. It is said to harden rapidly into a very hard mineral substance. The method of application is to wrap the pile in gunny-sack, or similar material, and then saturate it thoroughly with this cement in liquid form. This, of course, can be done by wrapping with one or more thicknesses of the gunnycloth, and when it is applied in this way it is said to form a hard and perfect protection against the entrance of the teredo or any other marine pest. The method of application is simple, and is said to cost about half as much as creosoting. The experience of two or three years, perhaps more, is said to show that it affords a perfect protection. If what is claimed for it is verified by experience, it will certainly be an important addition to the resources of the engineer in fighting marine pests. The speaker believes that it has been used along the Gulf Coast at several points, and that it has been used by the Louisville and Nashville Railroad, and upon some of the government works on the Florida Coast.

CORRESPONDENCE.

Mr. Collingwood

FRANCIS COLLINGWOOD, M. Am. Soc. C. E.—The paper is, altogether, the best exposition of the subject that the writer has seen, and is so complete that little room is left for discussion.

As to the depths at which these animals work, the writer wishes to mention the following instance. The site of the New York tower of the East River Bridge (at Pier 29, East River) was dredged to 31 ft. below high water. It was found that the dredge missed the ends of some piles that had been eaten off at about the mud line, say 30 ft. to 32 ft., and these were pulled up by chain attached by divers. Whether the worm was the teredo or not the writer cannot now remember.

The alarm about the caissons, mentioned by the author as existing at that time, arose from the fear that these animals might continue their work after the caissons were sunk in the bottom, even below the earth. After consulting many authorities, the writer reached the same conclusions as the author.

The paper is complete, and leaves nothing to be desired. Of course, the subject of remedies or prevention of the ravages of the worms is an extensive one, but it has been pretty thoroughly threaded over.

The writer's advice on this subject would be that patent nostrums should be avoided.

Mr. Le Conte.

L. J. LE CONTE, M. Am. Soc. C. E.—The Pacific Coast teredo is a very active mollusk, and when fully developed is ¾ of an inch in diameter and 36 ins. long. It is especially vigorous at and near the entrances to harbors where the flood tide waters come in fresh from the sea, and more particularly at those sites where the tidal currents are strongest and the food supply naturally plentiful. In such places the teredo will destroy a 14-in. pile completely in three months.

The teredo, as found on the Pacific Coast, is quite sensitive to the influence of fresh water encroachments, and as a result at different portions of the bay of San Francisco the difference in the activity of the teredo is very great—the life of a pile varying from three months to seven years. The worst experience with the teredo on the Pacific Coast has been the increase in its range of activity during the past ten years—the teredo now attacking greedily many kinds of materials which formerly were avoided. This apparent change in its habits is probably due to a variety imported by vessels plying to and from Mexican ports.

Some ten years ago the specifications of the Harbor Commission Engineer demanded that all piles furnished should have a perfect coating of natural bark; the trees to be felled in the winter months when the bark is more adherent. Now the teredo pays little or no

attention to the bark. The writer has seen several samples which were completely riddled with teredo holes. Mr. Le Conte.

The comparative freedom of the endogenous woods—palms and palmettos—from the ravages of the teredo seems to be due more to the very spongy character of the wood than to any other quality. This porosity readily admits the approach of the enemies of the young teredo before he is fully able to make a tube to protect himself. The natural enemies of the teredo should be studied and cultivated.

The *limnoria lignorum* mentioned by the author is a terrible pest on the Pacific Coast, and from a broad point of view is more destructive because of its much wider range of activity. The chelura has not been definitely recognized on the Pacific Coast by competent authority, although it may exist.

As to methods of protection, the outside coating, such as copper, zinc or the Paraffin Paint Company's coating are good preventives as far as they go; but by these processes the life of the pile is transferred to the natural life of its unprotected head, above the coatings, which generally decays so as to be unfit for service in about seven to eight years, sometimes longer. Dutch flat-headed nails have been used in San Francisco Bay by the United States Engineers, with marked success, against the teredo, but are of indifferent value as against the limnoria.

Creosoting properly done, with an ample amount of dead oil of good quality, is the best process yet devised as a protection against sea animals of all kinds, since it insures also against ordinary decay. The history of protective coatings for the past fifty years is, unfortunately, full of many inexplicable failures. Hence the slow advances made in practice. A study of these failures is quite interesting. A large percentage of them is charged to poor oil, whereas examination shows that such a charge is unfounded.

A large majority of these failures can be traced to the fact that it is very frequently impossible to impregnate a wooden pile with much uniformity in depth of penetration, some portions of the wood being close grained and hard, while other portions are much softer and more porous. Hence, during the time that the dead oil is being forced in under pressure it naturally penetrates through those channels which offer the least resistance to progress, and, consequently, most of the oil goes into the softer portions of the wood and avoids the denser portions.

It is evident that the quantity gauge, which keeps a record of the number of gallons of oil introduced per cubic foot, for the entire pile, can furnish no guarantee whatever as to whether the work done is first class or very inferior.

The writer has seen a creosoted pile, passed by inspectors as a good job, taken to a saw mill and cut up into short lengths, and the cuts

Mr. LeConte. examined. The inequality in depth of penetration was remarkable being in some places as much as 3 to 4 ins., and in the denser portions not over $\frac{1}{20}$ of an inch.

Along the west coast of Mexico the sea animals are unusually active, and piers are generally built on a foundation composed of wrought-iron piles 6 to 9 ins. in diameter. Iron and steel are being produced so cheaply that it begins to look as if this is the most practical solution of the problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 838.

DREDGES AND DREDGING ON THE MISSISSIPPI RIVER.

BY J. A. OCKERSON, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

WITH DISCUSSION.

The Mississippi River is utilized for purposes of navigation from the Gulf of Mexico to the Falls of St. Anthony, a distance of about 2 000 miles.

In this great length, the character of the stream goes through various changes, through the influx of tributaries and other causes. In order to get a clear comprehension of the controlling physical conditions in different parts of the stream, it is best to divide it into four distinct reaches or sections.

First.—That portion extending from the Falls of St. Anthony to the mouth of the Missouri, a distance of 712 miles. In this reach the banks are low, and the oscillation between high and low water rarely exceeds 25 ft. In the upper half of this reach the river is divided into a great many sloughs, which serve as high-water channels, but are often nearly or quite dry at low water. The water carries but little sediment; bank erosion is comparatively slight; for 21 miles it flows through a lake of slack water 30 ft. deep; the flow in two places is in-

terrupted by rapids where the bed of the stream is solid rock; in the upper portion, the navigable depth at low water sometimes gets down to 2½ ft., and navigation is usually suspended during the winter season for a period of four months or more in consequence of the river being frozen. The low-water slope averages about 0.5 ft. per mile. The low-water discharge is about 25 000 cu. ft. per second, and the high-water discharge about 350 000 cu. ft. per second. High water generally comes in May and June, and the low-water season usually begins about the first of September and lasts until navigation is closed by ice.

One of the rapids spoken of, near Rock Island, Ill., 1 575 miles above the mouth, has been improved by the removal of rock and the concentration of the volume by dikes and dams. The other, above Keokuk, Ia., 1 445 miles above the mouth, is surmounted by means of a canal, with 8 miles of slack-water navigation and three locks, with a total lift of 18 ft.

Second.—The second reach extends from the mouth of the Missouri to the head of the alluvial basin of the lower river, or practically to the mouth of the Ohio, a distance of 210 miles. This reach is the first to take up the enormous load of sediment put upon it by the Missouri River. Here permeable dikes are at their best, and immense deposits are easily induced where channel contraction is desirable to increase the depth. The banks are somewhat higher than those of the first reach, and the effects of bank erosion are more noticeable. The extreme oscillation between high and low water near the upper portion of this reach is some 36 ft.; the low-water slope averages 0.6 ft. per mile; the low-water discharge is about 45 000 cu. ft. per second, and the high-water discharge about 850 000 cu. ft. per second. At low water the navigable depths on the bars often reach as low as 4 ft. Overflows are not very frequent, as a conjunction of the floods of the upper Mississippi and Missouri Rivers is necessary to produce an overflow stage. The high-water stages usually occur in May and June, and the low-water season begins early in September and continues into the winter months.

Sand bars are numerous, and the crossings are consequently frequent, and their locations are constantly shifting. The river washes the rocky bluffs on one side or the other a greater part of the distance, and at Gray's Point, 1 100 miles above the mouth, it flows through a rocky gorge for a distance of 7 miles. At the lower end the

normal conditions are often complicated by back-water from floods in the Ohio, which causes the sediment to deposit, and this is a prolific source of annoyance to navigation.

This reach is sometimes frozen over for a month or so during the winter, but is more often free from ice throughout the year. The whole reach partakes much of the character of the Missouri River.

Third.—This reach extends from the mouth of the Ohio to the mouth of the Red River, a distance of 750 miles. Here the Ohio, reinforced by the Tennessee and Cumberland Rivers, comes in and controls the flood conditions of the lower river. The Missouri River no longer dominates, except in its mud; the bed of the stream is now through the deposits which it has built up and torn down repeatedly; at a few points the river is held in check where it strikes the bluffs, but for the most part the banks yield readily to the eroding power of the current. At one place a straight reach becomes excessively wide, as near New Madrid, by the river encroaching first on one bank and then on the other; at another place it becomes exceedingly crooked, as near Greenville and above, by the continued erosion of the concave bank and the building out of the point opposite the bend. The caving reaches enormous proportions, the maximum lying in about the middle third of the reach; the oscillation between high and low stages reaches over 53 ft. The banks are high; overflows are frequent; the sand bars are very large in extent; islands and towheads are numerous, and the width of the river here reaches the maximum; the bars which obstruct low-water navigation are not very numerous, and the depth rarely gets below 5 ft. on these bars, while by far the greater part of the reach has water of ample depth to satisfy all demands of navigation; the river is rarely obstructed even by floating ice; low-water conditions which interfere with navigation rarely exceed four months in duration, and now and then entire years pass without any serious interruption to navigation; the low-water slope averages about 0.35 ft. per mile; the floods usually come in February or March, and the low-water conditions from September to December; many of the bends have depths of over 100 ft., and the discharge varies from 65 000 cu. ft. per second at extreme low water to 2 000 000 cu. ft. per second at extreme high water.

The work of maintaining navigable channels through the crests of the bars by means of dredging has, so far, been largely confined to this reach.

Fourth.—The fourth reach extends from the mouth of the Red River to the Gulf of Mexico, a distance of 310 miles. In this portion of the river the channel is narrow and deep, the banks tolerably stable, and sand bars as obstructions to navigation are almost unknown. So far as navigation is concerned, this reach requires neither contraction works nor dredging. Nature has built a channel which man vainly tries to imitate in the reaches lying above. Only two islands exist in this reach, and the last gravel bar is near the upper end of the reach. The upper limit at low water is only 3 ft. above mean Gulf level, and the tidal effect is often observed over the whole length of the reach. The first practicable outlet, for the waters coming down, is through the Atchafalaya at the head of this reach. The extreme oscillation between high and low water at the upper end of the reach is 50.4 ft., and this, of course, tapers down to zero at the Gulf. The river in this reach carries the burdens of all the tributaries coming from a drainage area of 1 250 000 square miles. The destructive floods invariably come from the Ohio Basin, augmented at times by the tributaries below the mouth of the Ohio River.

The characteristics of the reaches described are pronounced, and they differ so widely as to make the improvement of each a problem by itself. The last-named reach should perhaps have terminated at the Head of the Passes instead of at the Gulf, 12 miles farther down. Where the current of the river meets the slack water of the Gulf of Mexico, bars are formed which require special treatment.

A general view of the conditions has been presented in order to make it plain that the problem involves different treatment in different reaches of the river.

Sand Bars as Obstructions to Navigation.—It is not easy to find a satisfactory explanation as to why sediment piles up in ridges instead of being distributed evenly over the bottom. These ridges of sand are usually found on what steamboatmen call crossings ; that is, on the path followed by boats when crossing from a pool lying in a bend along one bank to the pool in the bend of the opposite bank. These bars may be piled up to such an extent that during a high or even medium stage, their crests may be actually several feet higher than the surface of the water at low stage. The thread of the channel at high stages does not follow the low-stage channel, but crosses and recrosses it.

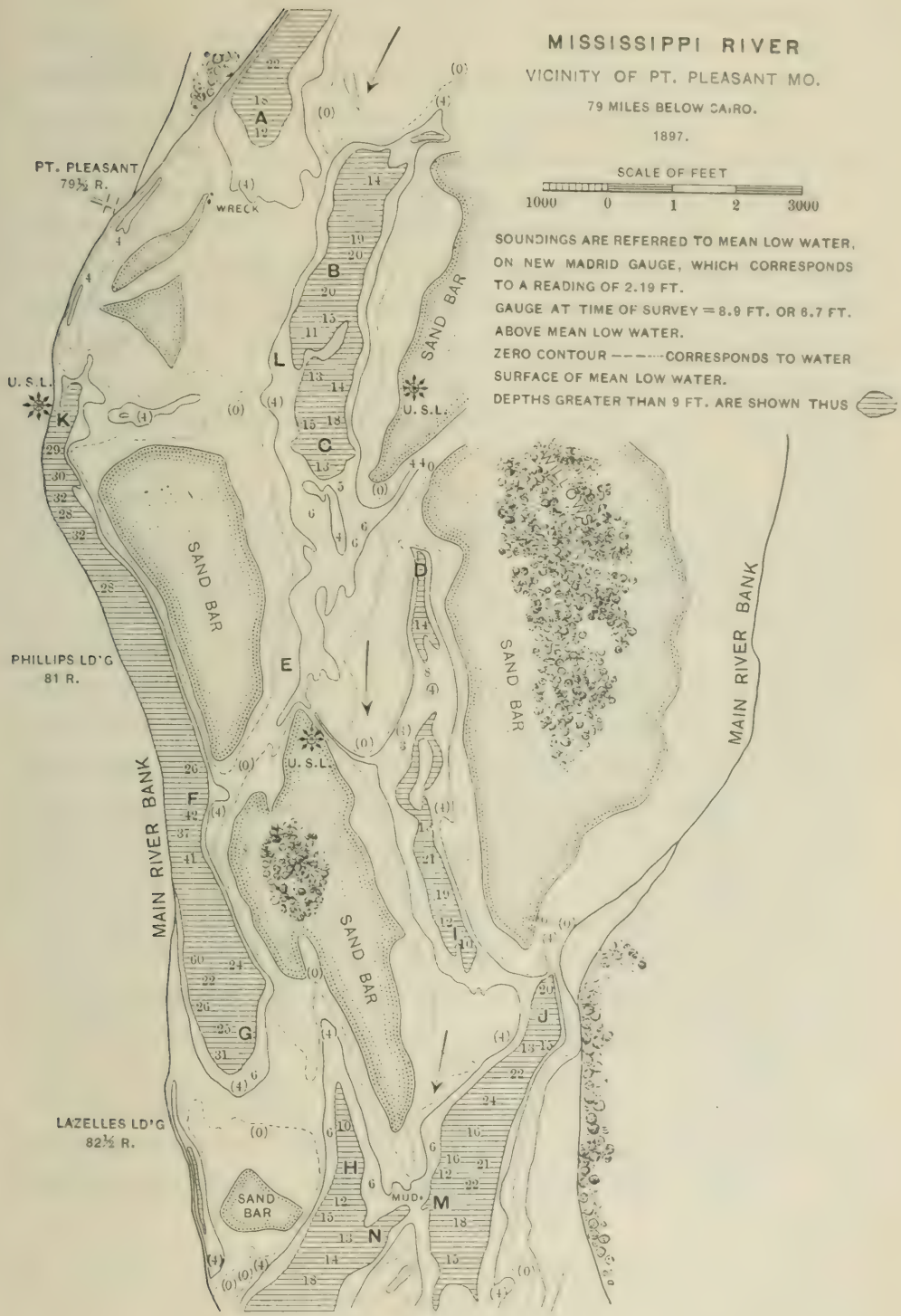

MISSISSIPPI RIVER
VICINITY OF PT. PLEASANT MO.
79 MILES BELOW CAIRO.
1897.
SCALE OF FEET
1000 0 1 2 3000
SOUNDINGS ARE REFERRED TO MEAN LOW WATER, ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.19 FT.
GAUGE AT TIME OF SURVEY = 8.9 FT. OR 6.7 FT. ABOVE MEAN LOW WATER.
ZERO CONTOUR ------ CORRESPONDS TO WATER SURFACE OF MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
PT. PLEASANT 79½ R.
WRECK
U. S. L.
PHILLIPS LD'G 81 R.
LAZELLES LD'G 82½ R.
MAIN RIVER BANK
SAND BAR
MUD
A
B
C
D
E
F
G
H
I
J
K
L
M
N

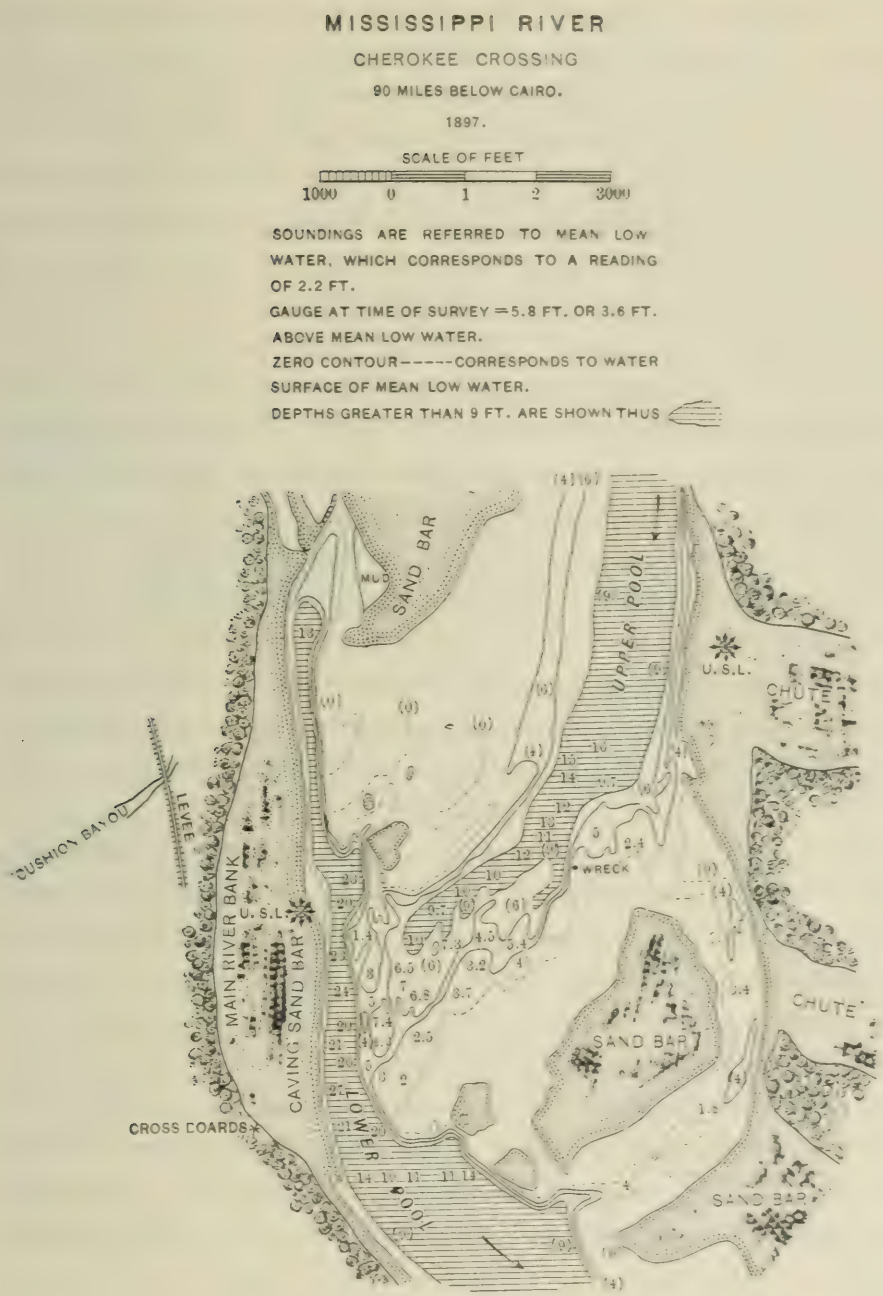
FIG. 1.

The first effect of a flood, with its increasing velocity, is to erode the bed and banks and add the material to the load already carried in suspension. This continues until the crest of the flood is reached and the decline sets in. The load is now too heavy for the diminishing velocity and the burden is rapidly deposited and obstructions are formed which, later, prove serious hindrances to navigation. When the river reaches a low stage these act as dams to hold the water in the pools. The slope on the crossing or dam is thereby increased, and likewise the velocity. The crest of the bar consequently cuts out, and if this cutting is confined to one channel a good navigable depth may be the result. If the bar is wide and flat there will probably be several insignificant channels, none of which answer the purposes of navigation. These must be concentrated by contraction or dredging, or both.

Fig. 1 shows rather a complicated case. This is near Point Pleasant, Mo., about 80 miles below the mouth of the Ohio. The general course of the river in this section is straight for several miles. The width is unusually great, due to erosion of both banks. The channel consequently is shallow and shifting. In Fig. 1 the pools are shown by parallel shading, and the figures indicate the depths. The upper pool comes down the right bank and terminates at *A*. The problem then is to reach the pool *B* over the reef which separates the two pools. When that point is reached, navigation to the lower end of the pool at *C* is easy. Then comes another crossing to *D*, followed by a narrow broken pool to *I*. Another crossing, and the fourth pool is reached at *J*. Here there is an abundance of water to *M*, then jumping a short reef into the deep pool at *N*, and this bad stretch of navigation is passed. Another route of more or less merit might have been found from the pool *B*-*C* at *L* to the shore pool at *K*, then following this long pool to *G* and over the reef to *H*.

Fig. 2 shows a simpler case and one most frequently met. The problem is simply to cross one reef lying between the upper and lower pools. Having crossed this, the depth is ample for several miles both above and below.

The problem then resolves itself into opening channels through the reefs between the successive pools. As a rule, the pools lie in the bends and the channel line crosses from one bank to the other. During the greater part of the year—say about nine months—the depth in


MISSISSIPPI RIVER
CHEROKEE CROSSING
90 MILES BELOW CAIRO.
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ZERO CONTOUR-----CORRESPONDS TO WATER SURFACE OF MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
SAND BAR
MUD
UPPER POOL
U. S. L.
CHUTE
CUSHION BAYOU
LEVEE
MAIN RIVER BANK
WRECK
CAVING SAND BAR
CHUTE
SAND BAR
CROSS BOARDS
LOWER POOL
SAND BAR

FIG. 2.

the river below the Ohio is ample for the requirements of navigation. During the other three months navigation is sometimes practically suspended in the third reach on account of shallow water. Whatever is done in the way of opening the reefs at such times answers for one low-water season only. The next flood probably changes the thread of flow to such an extent as to cross the opened channel and obliterate it completely. This is repeated with each high and low stage.

So far, the problem seems simple enough. The only thing necessary is to open a cut through the reef, composed principally of loose sand; but there are other difficulties familiar only to the engineer who has been taught by long observation and experience. While this artificial cutting is going on, Nature is doing some cutting on her own account, but on such a gigantic scale that the work of the largest centrifugal pumps seems insignificant. By the time the engineer gets what he considers a dredge of enormous capacity into position to open a cut from *A* to *B*, Nature perhaps changes her mind and opens a channel of her own down along the shore to *K* (Fig. 1); or, what would be worse, throws such a volume of deposit over the bar *A-B* that the dredged cut is filled as fast as the material can be removed. It is this tremendous volume of material that is moved along the bottom by the current, complicated by unforeseen, subtle changes in direction and force of flow, that often mocks the best efforts of the engineer.

That the difficulties of the problem are not generally understood or appreciated is manifest from the many simple solutions offered by inventors. Men who have never seen the river are prolific in devices for remedying all defects at trifling cost of time or money. They are by no means alone in the matter, for the men who have spent a lifetime on the river steamers also have ideas, sometimes very good ones, which they have developed and confided to the Patent Office. One inventor appeals to the Secretary of War for some "secret position," where he can study the currents of the river unobserved. He says he has a "nack of inventing," and also "has the nack of getting information." Furthermore, he believes there is some "sculluglery" going on which he might expose. He hesitates about giving his talents to the world, but says he is "willing to confide in the Government." He further says: "Necessity has been the mother of invention with me. Hydraulic engineers may oppose and discourage

my project and design, as it is calculated to do away with their work." With his device, "St. Louis would be a port for loading vessels direct from forring ports."

The necessity for some suitable device for the removal of sand bars has long been felt. Some thirty years ago a board of engineers recommended that a prize of $100 000 be offered for the best device for removing obstructing sand bars from navigable streams. Although Congress did not carry out their recommendation by making the necessary appropriation, a great many inventions have appeared

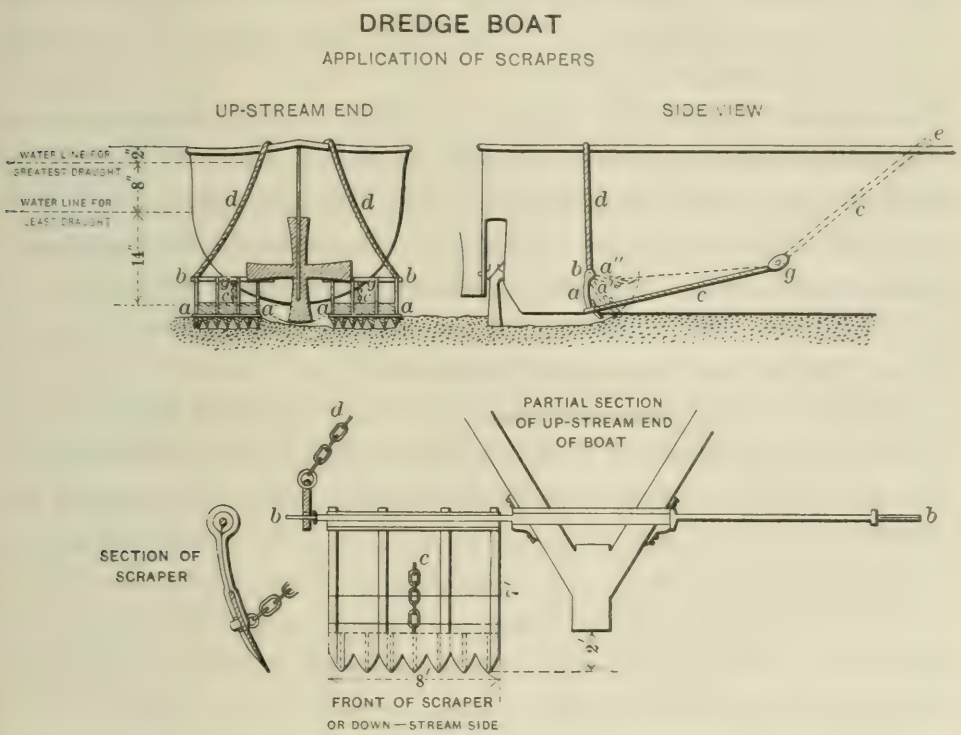


Fig. 3.

from time to time and have been tendered to the Government for use. A description of some of these may be of interest.

These devices can be divided into four general classes, viz., stirring and scraping, current deflectors, jets and the suction dredge.

Stirring and Scraping Devices.

(1) About the earliest application of this principle on the Mississippi was in 1867, when it was decided to improve Pass à Loutre by means of excavating and stirring or harrowing up the minute alluvial

material deposited from the heavily laden waters of the river. In this work a double-ended dredge boat, having an excavating screw with four blades 14 ft. in diameter, was used. This screw was similar to an ordinary propeller wheel and was similarly mounted. It was turned by means of a double engine at the rate of 60 revolutions per minute. It reached a depth 2 ft. below the keel. The work of the screw was made more effective by means of an auxiliary scraper attached to the up-stream end of the boat on either side of the keel (Fig. 3). The boat was moved down stream over the bar with the screw operating and the scrapers in position. In this way some of the bar material was again brought into suspension and carried off into deep water by the current.

During the first month's work with this dredge the depth was not materially improved. Later, better success was realized. In a little less than two months the depth was increased from 11 to 17 ft. The chief difficulty seemed to be in weak propeller blades, which were frequently broken and could only be renewed by docking the vessel.

This device was intended to cut out and maintain a 20-ft. channel through the bar at the mouth of the river.

(2) At Southwest Pass the same result was expected from the use of conical screws attached to the bow of a suitable boat. These cones were 20 ft. long and 5 ft. in diameter at their bases. They were set so that their points came together at the boat's stem and their bases were separated so as to cover a width of 20 ft. from out to out. Their axes were horizontal, the salient angle they formed being foremost. The flanges of the screws were 12 ins. wide at the base of the cones and diminished to 6 ins. at the points.

When these enormous screws were put in motion it was very difficult to guide the boat. The material was readily plowed up, but it was not broken sufficiently fine to be carried away by the current.

(3) In 1867 there was appropriated the sum of $96 000 for the construction and operation of two scrapers or dredges on the upper Mississippi, between St. Paul and the mouth of the Illinois River.

The first efforts made to remove sand bars by means of the scrapers invented by Col. Long was in the fall of 1867. These scrapers consisted of a frame attached to the bow of a boat and carrying a heavy cross-bar, to which were attached 6 steel buckets or cutters. This frame could be raised or lowered at will. In operating, the boat went to the

upper side of a reef, the scraper was lowered and the boat was then backed slowly down stream, scraping the sand with it down to deep water below the reef. This operation was repeated until the desired depth was obtained.

Two side-wheel steamers, the *Montana* and the *Caffrey*, were equipped with these scrapers by the Government, and, for a time, the steamboat owners operated a scraper boat between Keokuk and St. Louis at their own expense.

The *Montana* was 210 ft. long, 35 ft. beam and 5½ ft. depth of hold. She was equipped with two engines having 20-in. cylinders and 7-ft. stroke. The *Caffrey* was 150 ft. long, 30 ft. wide, 4½ ft. hold, with a draught of 32 ins. She had 15-in. cylinders with 5-ft. stroke.

This last boat was equipped and ready for work in October, 1867. Her first work was on a bar near Gray Cloud, 17 miles below St. Paul. Only 3½ ft. could be carried over this bar, and the regular packets could not cross it. After about four hours' work with the scraper the depth was increased to 3½ ft. entirely across the bar. The scraping was continued for two days and a depth of 4 ft. was secured. By November 15th all the bars between St. Paul and Prescott had been scraped and the depths increased to 3½ or 4 ft. At that date the packet companies notified the engineer in charge that the scraping had removed all obstructing bars and that no more work was required.

In 1868, when navigation became difficult, the scrapers were put into commission and worked throughout the season. They succeeded in deepening the bars from 8 to 18 ins., and this was generally accomplished with a few hours' work. Beef Slough was deepened from 3½ ft. to 4½ ft. in 35 minutes.

On the whole, the results were so satisfactory that steamboat owners announced that their boats had been making regular trips without interruption, "a condition of affairs never before known at this stage of river in the experience of pilots of 35 years' standing." The largest steamers had been able to reach St. Paul in the low-water season during two successive years, when without the aid of the scrapers they would have been obliged to tie up. Steamboat men said:

"These scrapers can be relied on to increase the depth over the crests of the bars at low water from 8 to 12 ins. This nearly doubles the carrying capacity of the boats now in use."

This scraping was continued for several years at a cost of about $20 000 per annum for each steamer, but as the relief was only temporary and had to be repeated from year to year, it finally gave place to the so-called permanent improvement, consisting mainly of channel contraction.

It should be borne in mind that in the portion of the river where the above-described scrapers were used, the obstructing sand reefs are quite short.

(4) An "excavator for water courses having current" consists of a horizontal shaft attached to the stern of a suitable boat, and fitted with a series of serrated metallic disks 40 ins. in diameter. Each disk has 29 cutters or teeth, which are turned alternately in opposite directions. These disks are revolved by the necessary machinery, at the rate of 100 revolutions per minute. The bar to be removed is cut out by running back and forth until the required depth is obtained. It is claimed that this device "is found highly efficient in raising deposits in channels and separating the earth into small particles so that it may be carried off by the current," and that the amount of work it can accomplish in a given time is "equal to that of 249 common bucket-dredging machines."

It is well to note that these claims are not based on actual trial in practical work.

(5) A "submarine plow," attached by means of timbers to the sides of a boat, aims to plow out a furrow in the bar. Spiral springs are attached to the timbers holding the plow so as to force it down to its work, and so that it will adapt itself to any irregularities of the bottom. The efficiency of the plow is increased by water-jets forced into the furrow for the purpose of breaking up and scattering the deposit moved by the plow. Beyond this, the current is relied on to carry the deposit to deep water.

(6) A "channeler," consisting of a horizontal revolving drum made of boiler iron, with a series of plows and scrapers attached to its convex surface, is conspicuous among the earlier inventions. The revolving drum is 40 ft. long, and is suspended by journals near its extremities. It is revolved by means of two 35-H.-P. engines, which operate the drum through gear wheels acting on a large cog wheel encircling the drum half way between the ends.

The scrapers are set spirally on the drum for the purpose of carry-

ing the material outwardly each way from the middle. When the material reaches the end of the drum it is supposed that the current will take it up and carry it away, leaving a clear cut equal to the length of the drum. The drum and its shaft are attached to the stern of a boat. In this case the attachment is rigid, and the depth is adjusted by means of water-tight compartments in the boat which can be filled and pumped out at will.

In operating this device the boat is dropped down on a bar with bow end upstream. The drum is then made to revolve as the boat moves down on the reef. By means of anchors and lines attached to windlasses, the boat is drawn stern foremost down across the bar as rapidly as the scrapers on the drum loosen up and remove the deposit. It was supposed that the excavated material would be thrown from the ends of the drum with sufficient force to carry it entirely free from the cut, and it was estimated that in sand this device could make a cut 40 ft. wide and 3 ft. deep at the rate of 150 ft. per hour.

In moving from one bar to another the drum is used to propel the boat, which in this way, it was claimed, could travel at the rate of 8 miles per hour. The estimated cost of a craft of this kind was $25 000. A crew of 12 men was required to operate it.

Like many others, the merits of this device were never tested by actual work on sand bars.

(7) Another "scraper" consists of two arms, each 80 ft. long, placed parallel to each other and hinged at their middle points on the stern overhang of a side-wheel steamer. At the outer extremities of these arms is fastened a cross-beam, to which is hinged the scraper backing, which is rectangular in shape and made of boiler iron properly stiffened. Several scoop-shaped scrapers and plows, placed alternately, are attached to the lower edge of the backing. From each end of this cutter a chain leads to the steamer, for the purpose of taking up the strain and regulating the inclination of the scrapers. The scraper is raised or lowered by tackle leading from the inboard end of the arms to a drum on the deck of the steamer.

With this scraper the boat moves over the bar from its up-stream side and with head down stream. The sand is thus scraped down to the deep water. The operation is repeated until sufficient depth is obtained.

(8) "A steam circular sand bar dredger" was brought to public notice in 1878. This machine commended itself on "account of its practicability." With it, the inventor says:

"There is no need of costly engineering and surveys, no examinations to be made and no commissioners to report on what is necessary to be done. As a sand bar dredger it is invaluable; commencing at the lower edge of a sand bar by lowering the wheel to the required depth, a channel may be cut through in a remarkably short space of time, the current of the river carrying away the sand displaced by the revolving toothed wheel."

This machine was also intended for opening channels through ice. It was claimed that it could cut its way through solid ice 2 to 3 ft. thick at the rate of 4 to 6 miles an hour, and thus open a channel wide enough for steamboats.

"The harbors of the world heretofore closed by ice from 3 to 5 months in the year can be kept open so that ships can go to sea every day in the year." "One of these boats on each canal level would keep them open to commerce during the winter."

Offers were made to "guarantee" 10 ft. of water in the channel of the Mississippi River from St. Louis to Cairo in the driest season, with one "dredger" costing $40 000.

"The advantages of this invention to the commerce of the world is beyond computation."

(9) Another scraping device consists of a steel hull with a wedge-shaped prow, with auxiliary wings attached. This was intended to act something like a huge snow-plow, the wings or deflectors being designed to carry the loosened material far out each way from the cut. Jets were also provided to loosen up the material.

In actual operation the hull is sunk to the bottom by admitting water, and is then pulled up stream by powerful towboats. The wedge-shaped prow buries itself in the sand, which is pushed along as the boat is moved, and finally finds its way out to the ends of the deflectors. In moving from one bar to another, the boat is first floated by pumping the water out. This is a comparatively new device.

(10) Another comparatively recent device consists of a gigantic harrow having a water-jet at the extremity of each tooth. The pressure chamber forms the frame of the harrow. This harrow was to be attached to the bow of a boat and be provided with suitable rigging and winches by means of which it could be raised or lowered at will.

(11) A different design, or, as the inventor calls it, "Means for deepening the channels of rivers," consists of four "circular plows," or agitators, revolving on vertical shafts passing through the hull and operated by the necessary driving engines and gearing. Each screw can be raised or lowered independently, as required. These screws are intended to loosen up the material, and the natural current is supposed to do the rest. The forward pair of screws are smaller than the others, for the purpose of first opening a small channel, which is widened by the two which follow. The boat is moved back and forth until the required depth is reached.

It was claimed that four boats equipped in this way could make and keep open a channel 10 or 12 ft. deep between St. Louis and Vicksburg.

(12) An "excavator," similar to that described in Section 10, is attached to the bow of a boat, and has vertical shafts with scrapers at the lower end which are revolved by means of belts. The depth of the shafts is regulated by a screw at the upper end of each. This device also relies on the current to carry off the loosened material. The inventor says that "The revolving cutters cut up the material and churn and stir it to a considerable extent, so that it will be carried away by the current."

(13) Another "channel plow" inventor has a device which he claims will produce:

"A mean smooth bottom, circular in form and non-resisting, which will be of sufficient width and depth to contain and carry off within banks the greatest floods, resulting in the disappearance of all snags, sawyers and bars."

It will "Provide a steady flow of tranquil waters at all seasons and stages." It will cause a "Total cessation of overflows and the consequent reclamation of many river-bottom lands, and that, too, without the aid of levees."

This, certainly, covers the whole ground. The earnest efforts of all hydraulic engineers interested in river improvement are directed toward the accomplishment of the results set forth, but none of them are as confident of success as the inventor seems to be.

(14) One of the latest devices offered, to solve the problem of removing sand bars, is made in the form of a conveyor such as used in mills and elevators, but, of course, much larger in size. This spiral or screw has blades several feet wide, attached to a hollow shaft, and is made up in sections 10 ft. in length. These sections are strung on

a steel spindle until the whole screw is of sufficient length to reach across the bar. To utilize this device, it is dropped down over the bar, and, when in proper position, it is held in place by means of anchors or piles connected with wire cable to the upper end of the spindle. It, of course, trails down stream parallel to the current, and the pressure of the current against the blades causes the screws to rotate. In this way the sand is loosened up and conveyed to the pool below the bar.

All the power required in this case is derived from the current, and as the screws themselves are comparatively inexpensive, this would be a very economical method of cutting down bars if the inventor's ideas of efficiency were realized.

(15) A simple arrangement, intended to make each boat independent of outside aid when shoal water is encountered, is that in which the hull of the boat is provided with wings, which, when not in use, fold closely against the sides. On approaching a shoal the wings are opened and a portable dam is thus formed which raises the water; and this, in addition to the pressure on the wings, enables the boat to ride safely over what would otherwise be an impassable barrier.

(16) The "automatic sand dredger" consists of a stern-wheel boat to which is attached a horizontal shaft bearing a number of cutting disks, constituting a dredging wheel. The extremities of this wheel shaft are connected by bars to a shaft extending across decks near the bow of the boat. A convenient hoisting drum serves to raise and lower the cutter. The cutter has considerable weight, and the hooked cutters penetrate the sand, which is loosened and lifted up where the current catches it. As the boat is moved over the bar with the cutter lowered, the cutter wheels roll along the bottom.

(17) Another scraping device consists of a suitable boat provided with two long parallel booms or spars across decks about amidships. These booms project about 50 ft. out from the sides of the boat. The ends of the booms are braced and are supported by guy lines leading to a central mast, and also a second pair of guys leading to the bow and stern of the boat. The wings are hinged on each side of the boat to the forward boom. They extend from the side of the boat out to the end of the boom, and are wide enough to reach the bottom when let down on a bar which it is desired to deepen. The lower edge of these wings is weighted and provided with teeth or plows which pene-

trate and stir up the material. Guy lines, leading from the lower edge of the wing to winches on deck, serve to hold the teeth in contact with the bottom. When not in operation the wings are lifted up flush with the deck by means of tackle attached to the rear boom.

In operating this device, the boat moves down stream over the bar, carrying the sand with it over the reef. The current acting on the wings helps the boat along. This operation is repeated until the desired depth is secured.

(18) Another device is a "dredging machine" for use in "working out the channels of rivers and in removing sedimentary formations from the beds of running streams." The agitators are called "wipers." These agitators are attached to the stern of a side-wheel boat and are operated by means of bevel gears so arranged as to revolve the wipers in opposite directions. They are hung on a cross-shaft which admits of their being raised or lowered. The wipers are made with spiral flukes, so that the lower points will enter the deposit first and then draw it upward, thus lifting it from the bottom, and disintegrating it so that the current can carry it away. The number of agitators will depend on the magnitude of the work. The boat is to be moved back and forth over the bar, with the agitators at work, until the required depth has been obtained.

(19) A ship's drag for dredging rivers is a device to be attached to a steamboat or barge and dragged over the bar to be deepened. The teeth rake and stir up the bottom to such an extent that the current carries the material away. The teeth of the drag are rounded in front and have a groove in the back. The object of this groove is to form an eddy and thus aid in keeping the matter in suspension until the current takes it up. The frame holding the teeth can be separated into several jointed parts by means of hinges, and in this way the scraper will conform to the shape of the ground over which it moves.

Current Deflectors.

(1) The most pretentious of current deflectors is a "Machine for Deepening River Channels," invented in 1879. The object of this machine is to "deflect the current of a river downward, and thus cause the said current to deepen the channel." This consists of a boat to which is attached a triangular box which can be lowered to a point near the bed of the stream, the axis of the box being at right angles to

the boat and to the current. The up-stream edge of the box has flanges for regulating the flow.

In operating this machine it is moved across the bar, either up or down stream. The current strikes against the inclined side of the box and is thereby deflected downward, which causes it to impinge against the bed of the river and wash away the sand to a considerable depth. In case the bed is composed of clay not easily abraded, it is first loosened by means of a revolving toothed cylinder, and then carried away by the deflected current. The boat is held in position against the current by means of arms pivoted to the sides, the lower ends resting on the bottom. A wheel in front of the device is of sufficient diameter to show above water when resting on the bottom. The object of the wheel is to bring up samples of the bottom of the river so that the "engineer may know the exact kind of bottom there is at each point," and conduct his operations accordingly.

The device is further provided with a graduated disk attached rigidly to the arms supporting the triangular box. To the center of the disk is pivoted a weighted pointer which will always hang vertical and hence show the inclination of the box supports and consequently the depth of water. The inventor also provided an automatic recording device which would plat the profile of the bottom as the boat passed over it.

In moving from one bar to another the deflecting box is to be raised above the surface of the water by means of lines attached to the rear arms. Just how this is to be accomplished is not clear.

(2) Another device for deflecting the current downward on the bed of the stream is styled a "dredger," and is described as a "simple, inexpensive and efficient device for deepening the channels of rivers, removing sand or mud therefrom and for preventing the formation of sand bars." These desirable results are accomplished by means of several parallel plates of different lengths which are set at an angle with the vertical, but having the lower edge of the plates in the same plane. These plates are securely held in a suitable frame, and the whole is supported on short legs.

This frame, with the attached plates, is set on the bottom of the stream with the plates inclined up stream. The current, in striking the plates, is deflected downward, and thus produces a scouring effect on the bottom. When sufficient depth has been secured it is moved

to a new position. The size and number of these devices to be used is, of course, governed by the magnitude of the channel desired. The device is to be handled from suitable boats or barges. A device similar to this was used in the improvement of the South Pass with some success.

If it is desired to erode a bank the device is turned on edge so as to throw the current against the bank. To prevent the formation of sand bars, this inventor says that all that is needed is to suspend near the bottom V-shaped plates with perforations at the angle. The current caught by the open arms of the plate will escape through the perforations with force enough to keep the sand on the move.

(3) Another "device for regulating the flow of streams," deepening the channels and removing obstructions is also intended to deflect the current downward against the bottom. It consists of a frame having a suitable number of cross-bars to divide the interior space into cells of proper size, depending on the magnitude of the stream. The upper surface of the frame and cross-bars are filled with teeth, which, together with the cells, obstruct the even flow of the water and produce a "violent suction or eddy in the current." This increases the momentum or gravity of the water and causes it to "force itself violently" through the orifices between the cross-bars and thus churn up and wash away the bed of the river. In this way the sediment composing the bottom is agitated to such an extent that the stream is compelled to take it up and carry it away in suspension.

The device can be made in any number of sections joined together by chains and anchored to the bottom of the river. In this way, it is said, bars can be prevented from forming and the bed of the river can be depressed to any desired extent.

It is perhaps well to note here that the principle involved in the foregoing devices, and which is relied upon to erode the bottom, is to be found in several other devices where it is expected to induce deposit. If the current can be deflected downward and erosion produced, then a barge anchored over a reef should produce similar results, as the current striking the end rake would be deflected downward. In practice, it is a well-known fact that a barge anchored in shallow water will very soon cause the sediment to deposit. In dredging, the plant is never left in the cut for any great length of time when the pumps are not in operation. If a break-down occurs which necessi-

tates extensive repairs, the plant is swung out of the cut so that the current may have free access to it.

Nearly all the current deflectors proposed are intended to deflect the current laterally, and for this purpose inventors have brought out jetties of various kinds.

(4) A simple and efficient jetty deflector is made by driving piling along the desired line at intervals of about 20 ft. On the up-stream side of the piling small flat boats 16 by 40 ft. in size are lashed in a continuous row. Corrugated steel aprons 10 x 10 ft. in size are hung over the upper side of the flats at a considerable angle with the river bed, to allow for straightening up as the bottom scours out. The corrugated sheets of steel are stiffened by three angle-bars of 3 x 3-in. section firmly riveted to each sheet. The jetty is also constructed without the boats by attaching to the piling a stringer on which the plates are hung.

This jetty has been used with success between the Missouri and Ohio Rivers. Twelve hundred feet of jetty was built in one locality in four days with a force of twenty-two men. It cost about $2 per running foot. The effect was almost immediate. The water was deepened from 4 ft. to 7 ft. with a fall of 1 ft. in the stage of the river.*

A similar jetty has been proposed with the substitution of canvas for the steel plates and omitting the use of the flats. In this device it is proposed to make a curtain of canvas, say 15 ft. wide and 100 ft. long; one edge and the seams to be reinforced by sewing in hemp rope of suitable size with good lashings at the edge. At the other edge a chain is to be lashed to the canvas as a sinker. The whole is rolled up like a reefed sail. In order to use it, the piling is driven as before. The roll of canvas is stretched across the up-stream side of the piling above water and lashed at each pile. When everything is ready the reef lashings are to be cut and the weighted edge causes the canvas to unroll and sink to the bottom. The flexibility of the chain insures a close contact with the bottom. As many lengths of curtain can be used as may be found necessary.

(5) Similar deflectors are made portable by means of double-hull boats which admit of dropping sheet-piling down between the hulls. When it is desired to deflect the current, or cut off a side chute, a num-

* Report of Chief of Engineers, U. S. A., for 1896.

ber of these boats are placed in position, end to end, until sufficient length is obtained. These are known as the King jetties.

(6) Another way of accomplishing the same result is by means of steel barges of suitable length, known as the Marsh jetties, which are designed to sink by filling with water. The ends are wedge-shaped and the sides are straight, so that one boat may be lapped and fastened to the next, and so on until sufficient length has been obtained. In this way the current will be effectually deflected to the channel which requires deepening.

When the desired results have been obtained the water is pumped out of the hulls, which are thus floated and are then ready for another bar (see Fig. 4).

(7) Another inventor, somewhat earlier than the last named, hit upon a wooden sinkable deflector and plow, by means of which the "Sand,

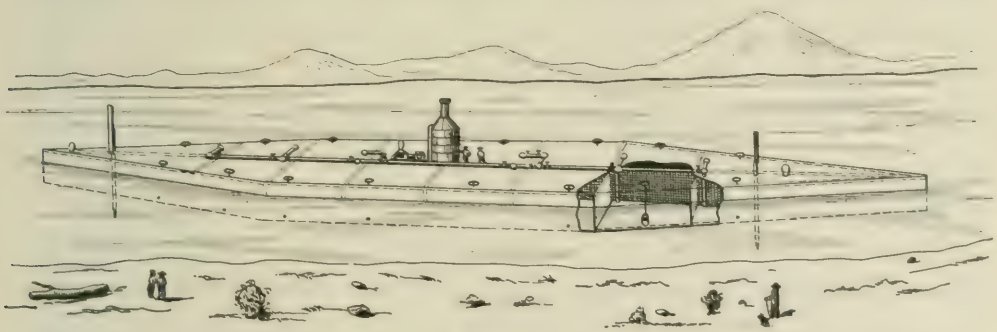


FIG. 4.

sediment, etc., at the bottom of the river will be sufficiently irritated to enable the current to carry it off." The jetties consist of strong wooden boats triangular in cross-section, the apex of the triangle forming the keel. These boats are designed to be from 500 to 600 ft. long. The plan would also be triangular, the greatest width being in the middle and then tapering off toward each end, which terminates in a strong timber head. The upper portion is open, with occasional cross-bracing. In order to use this jetty it is towed to the point desired and then sunk by filling it with water.

In order to secure the required length several may be sunk with a slight overlap. In this way the current is supposed to be deflected so as to cut out bars, or even make a cut-off if required. When the work is done the jetties are floated by pumping out the water. It is claimed that they can be "raised and removed in two hours."

An attachment termed a plow is applied to the towboat which handles the jetties. Its function is to loosen up the hard material so that the current from the jetties can get hold of it and carry it away. This plow can be raised or lowered at will, by suitable winches which are provided for that work. It is claimed that by the use of these devices rivers can be plowed to a depth of 30 ft.

(8) Some inventors have directed their energies toward the use of material at hand. One of the earliest of this kind of deflector was made of huge trees with branches intact, and sunk in the desired place by means of heavy weights attached to the butts, while the tops were free to take the direction of the current. A row of such trees are sunk side by side, and, if need be, others are added on top. This is not intended to stop the flow to such an extent as to form eddies, but sufficient to silt up the stream at or below the dam and deflect the water to the desired channel, and cut out the bar against which the current is directed.

(9) Another inventor uses similar trees in a different manner, which he claims "Is founded on natural and therefore correct principles, to correct, widen, deepen and confine the channels of the rivers."

In this system the trees are sunk as before, but the tops must be placed up stream to catch the small drift and sand, which very soon accumulates and fills up the dike. The trees may be held in place with piling. This jetty is intended to induce rapid deposit, which soon forms around the trunks of the trees, "Firmly anchoring them to the bottom ; the branches are next surrounded and covered by sand, completing by a natural process the indestructible and impregnable dam."

Water-Jets and Auxiliary Devices.

Several jet devices have been proposed which aim to make each steamboat independent of any general improvement of the channel by providing suitable jets which will enable the boat to work its own way through the bars.

(1) One of these jet devices consists of two double-acting pumps which force water through twenty-eight 3-in. nozzles arranged symmetrically about the bow of the boat. On reaching a shallow bar, the force pumps are started, the jet valves are opened, and the boat is pushed ahead as the sand yields to the eroding force of the jets. In this way

the boat carries a pool with her across the reef, and at the same time will probably improve the depth for the boat that follows.

(2) Another device aimed to accomplish the same result with a different kind of pump and a different arrangement of the jets. It was claimed for this invention that its use would make channel improvements with dredges, contraction or other means entirely unnecessary. By this scheme each steamboat was to be equipped with a centrifugal pump with a capacity of about 2 000 galls. per minute. The discharge pipe was designed to run the whole length of the hull along the center truss. The jets emanating from this pipe were arranged to discharge along the lower line of the hull directly on the sand. The jet nozzles were arranged so that they could direct the stream either forward or backward, so as to always act with the current, whether the boat were going up stream or down.

When the boat reached a bar the pump was to be started and the jets opened, the boat being constantly forced ahead until finally carried over the reef. The estimated cost of this attachment was only $3 000 for each boat, while the cost of delays in a single trip caused by grounding might readily exceed this amount.

In urging the utility of this method the inventor makes the following statement :

"It cannot be expected that the Government will every year spend thousands of dollars to remove sand bars which re-form at every flood. Those interested in river navigation ought to make themselves independent of such obstructions and of Government aid, but they will not entertain the idea until forced to."

Admitting that these devices would enable a single boat to force its way through obstructing sand bars, there still remains the great bulk of traffic, which moves in barges that cannot be carried over by the methods proposed.

(3) Another jet device, intended to remove the obstructing bar, is called a "Submarine plow or hydraulic apparatus for removing sand bars." It consists of several jet pipes fixed to a drag frame which is pivoted to the bow or stern of a suitable boat. Each pipe terminates in three jets, one of which points downward and loosens the material, while the other two are bent so as to pick up the loosened material and carry it upward where the current will take it and move it along. The combined force of the current and the jets carries the particles of sand some distance before they come to rest. The curve of each jet

pipe is filled in with a web, so that if it strikes a log or other obstruction it will slide over it without catching. The whole device is provided with tackle and winches conveniently arranged to raise or lower the jets, and suitable pressure pumps supply the necessary water.

It is claimed that a vessel with a 500-H.-P. equipment of this kind can erode a depth of 5 ft., taking out a cross-section of 27 sq. ft., at the rate of 6 miles an hour, throwing the sand and gravel to the surface in 30 ft. of water.

This device was used in Gedney's Channel, New York Harbor, but was abandoned on account of the slack current at ebb tide and the influx of material at flood tide.

(4) Water-jets were used with marked success on Horse Tail Bar, near St. Louis, in 1881. Four pile-drivers were lashed two abreast with their heads together. Each one of these drivers was equipped with a Worthington duplex pump with a $7\frac{1}{2}$-in. steam cylinder and a $4\frac{1}{2}$-in. water cylinder having a 10-in. stroke. The capacity was about 165 galls. per minute each. The four jets were brought together. Anchors were laid and the drivers were pulled back and forth across the reef with a steam windlass, all the pumps working in the meantime to their full capacity. In ten hours' actual work in this way the channel was deepened from 6 ft. to 8.3 ft. for a width ample to pass the largest tows.

(5) In 1896 a jet dredge was constructed under the supervision of Major Thos. H. Handbury, for the work between St. Louis and Cairo. The main features are two 15-in. centrifugal pumps driven by direct-connected 15 x 14-in. engines. The steam is furnished by two 4-flue boilers 28 ft. long with 42-in. shells. This machinery is mounted on a suitable barge.

The suctions lead over the sides of the barge, and the discharge leads over the bow. The capacity is 20 000 galls. per minute. This water is forced, through four 12-in. discharge pipes with flattened nozzles, against the sand, which is thus stirred up and washed into deep water. The attack is from the up-stream side of the bar, and the boat is dropped down, with jets in operation, until the lower pool is reached. This operation is repeated until the desired depth is obtained. The plan and elevation of the bow of this jet dredge are shown in Fig. 5.

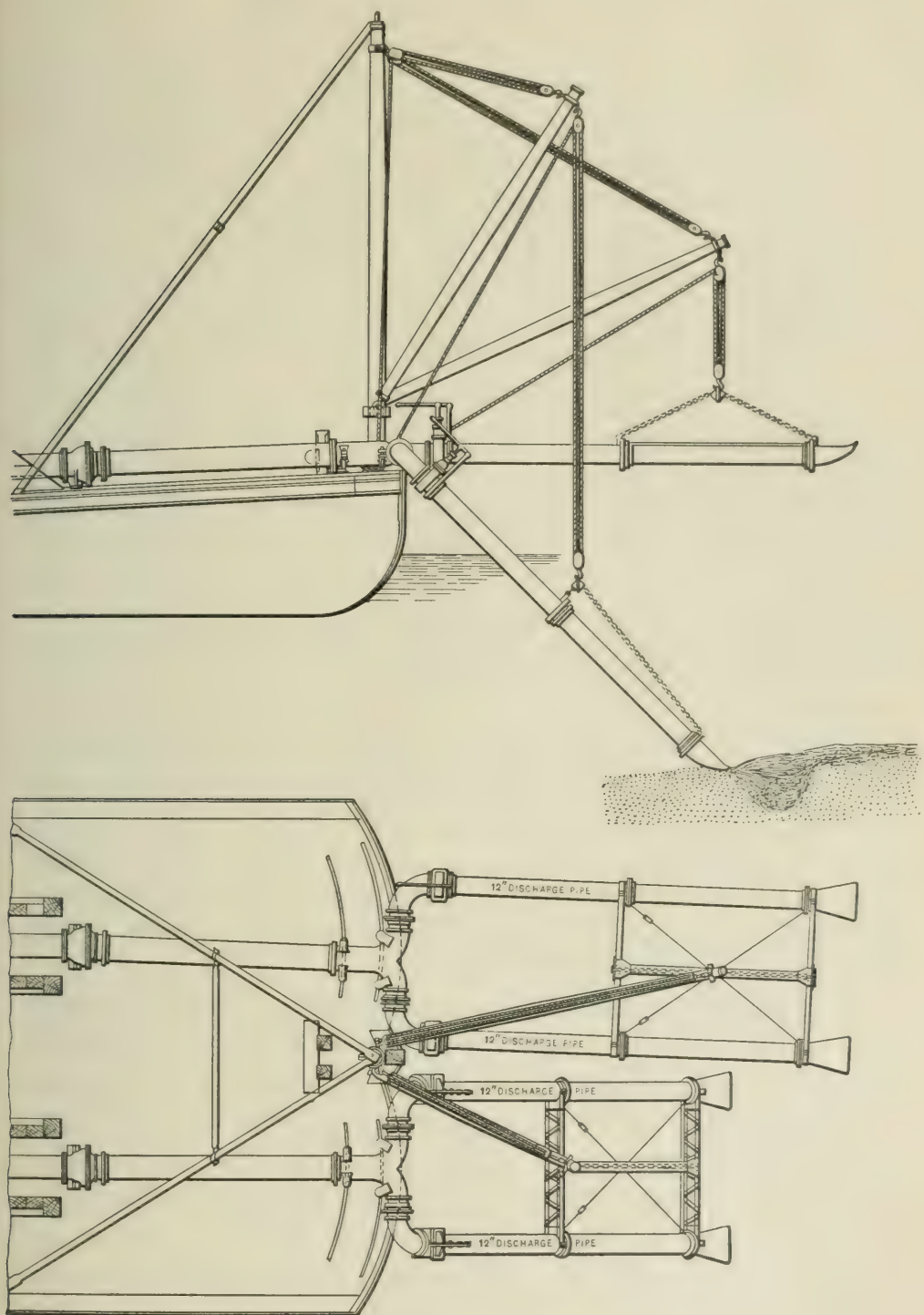

12" DISCHARGE PIPE
12" DISCHARGE PIPE
12" DISCHARGE PIPE
12" DISCHARGE PIPE

FIG. 5.

On short reefs, this device is reported to be quite efficient. Where the reefs are of considerable length the accumulation of sand in front of the jets becomes too great to be moved economically in this way.

(6) By far the most pretentious improvement scheme by means of jets, yet proposed, is the "Adams Flume," the construction of which was authorized by Congress in 1879. The purpose of this device was to "Establish a permanent channel in the Mississippi River from St. Paul to the Gulf of Mexico," a distance of some 2 000 miles.

It consisted of a triangular iron tube with sides of about 12 ins. The different lengths of this pipe were provided with flanges at both ends, so that they could be bolted together, a rubber gasket being placed at each joint to give the pipe sufficient flexibility to adapt itself to the undulations of the bottom and the curvature of the channel. The pipe was divided longitudinally into two compartments by means of a metal partition. The lower compartment was provided with $\frac{1}{2}$-in. jets at intervals of 1 ft. Clear water was pumped into the upper compartment under a pressure of 5 to 8 lbs. per square inch. By means of suitable valves the water could be let into the lower compartment and through the jet pipes; then, in the language of the inventor:

"It stirs up the sand, holding it in suspension till the current carries it off to some low place where it forms banks like a canal. The number of jets to be opened at any time is governed by the pressure. Where there is any obstruction it is opened on like a battery; no sand, gravel or moderate hard-pan can stand near it. To lay this pipe in one unbroken line in the center of the river from its head to the Gulf, it will scour a channel the necessary depth and width throughout and keep it open all through."

In this work one small Worthington pump was considered sufficient for each 100 miles of pipe. Clear water was said to have greater eroding power than water containing sediment, and in order to secure it, the river water was allowed to flow into a tank or well attached to the boat and the pump took its supply from the well. Just how the sediment was to be eliminated by this process is not altogether plain.

More than eight years and about $40 000 were spent in attempts to prove the practicability of this device, but the project was finally abandoned without ever laying a foot of pipe in the river channel. This large waste of time and money cannot be charged to the engineers, for Congress, through the Secretary of War, dealt di-

rectly with the inventor, the law requiring that the device should be made and tested under his supervision and direction. For five years the "preliminary arrangements" were going on, the expenditures having reached the sum of $27 500. One locality after another was agreed upon as suitable for the trial, but there was always something to prevent an actual test. Another appropriation of $15 000 was made, and the "preparations" were continued for several years more without results of any kind. A board of engineers was then appointed to examine and report on the advisability of continuing the experiments. Their conclusions were that it could not be successful even on a small scale.

The engineer officer in charge of the district was instructed to take charge of the work pertaining to the flume. The only thing found to show for the $40 000 was a small amount of sheet-iron pipe made for experimental purposes, and little or none of this was completed and ready for use.*

(7) The removal of bars by means of blasting has been tried, but with unsatisfactory results. Torpedoes, loaded with 75 to 160 lbs. of powder, have been exploded at depths of 10 to 20 ft. below the surface of the sand, but the results have invariably proved disappointing. Large quantities of powder in tin canisters laid on the crest of the bar have been exploded without materially improving the channel depth.

In the foregoing pages the author has attempted to give a brief account of various devices which have been offered to solve the difficult problem of improving the low-water navigation of the Mississippi River. A few of the devices have been actually used, and still fewer have been moderately successful.

The Long scrapers used on the upper Mississippi some 28 years ago were pronounced satisfactory, although the best they could do was to increase the depth over short reefs 12 to 18 ins. About this time the use of temporary expedients gave place to a system of permanent improvements, consisting of the closing of chutes, the contraction of wide places by means of jetties or dikes, and the revetment of banks. The inauguration of this policy, to the exclusion of temporary expedients for channel improvement, was evidently a mistake, due probably to the fact that the length of time required by the so-

* Report of Chief of Engineers, U. S. A., 1888, p. 1488.

called permanent method before tangible results could be realized was very much underestimated.

In the meantime, competition of railway lines along the banks of the river, and difficulties of navigation, threatened to annihilate the river traffic. Steamboat men saw their profits dwindling, their business gradually dying out, and they urgently appealed for relief. They could not wait for the completion of permanent improvements. Immediate temporary relief must be found, or the river as a great commercial highway must be abandoned. It was in response to this appeal that temporary channel improvements were again taken up and were finally, after several years of agitation, made obligatory by Congress, for the lower river, in the River and Harbor Bill of June 3d, 1896.

In the expedients which were tried or suggested for temporary improvement of the channel, and which have been described in the preceding pages, the chief defect lay in relying on the current to carry off the material after it was loosened up. It was generally thought that if the crest of a bar was thoroughly loosened up, the current would carry the material away, and thereby the depth would be increased. Engineers very soon saw that but little additional depth could be gained in this way.

River men still talk of a "crust" on a sand bar, and believe that it prevents erosion by the current until broken by scrapers or other means, and that after this is done erosion is rapid and effective. One of the members of this Society was probably among the first to recognize the necessity of carrying the spoil out of the channel by artificial means. In 1870, after experimenting with dipper dredges for some time, Robert E. McMath, M. Am. Soc. C. E., reached the conclusion that:

"The successful use of sand pumps and hydraulic conveyors and distributors of débris inclines me to the opinion that a combination is possibly better adapted to the work of river improvement than any means yet used."

In 1883 a plant was provided for dredging in the Mississippi River above the mouth of the Illinois River. This plant consisted of an Osgood dipper-dredge and six dump-scows, the whole plant costing $29 348. It was used with good results. The first work was done at Howard's Bar, near La Grange, Mo., where a channel was opened 1 000 ft. long, 110 ft. wide and 5 ft. deep. It was completed in about

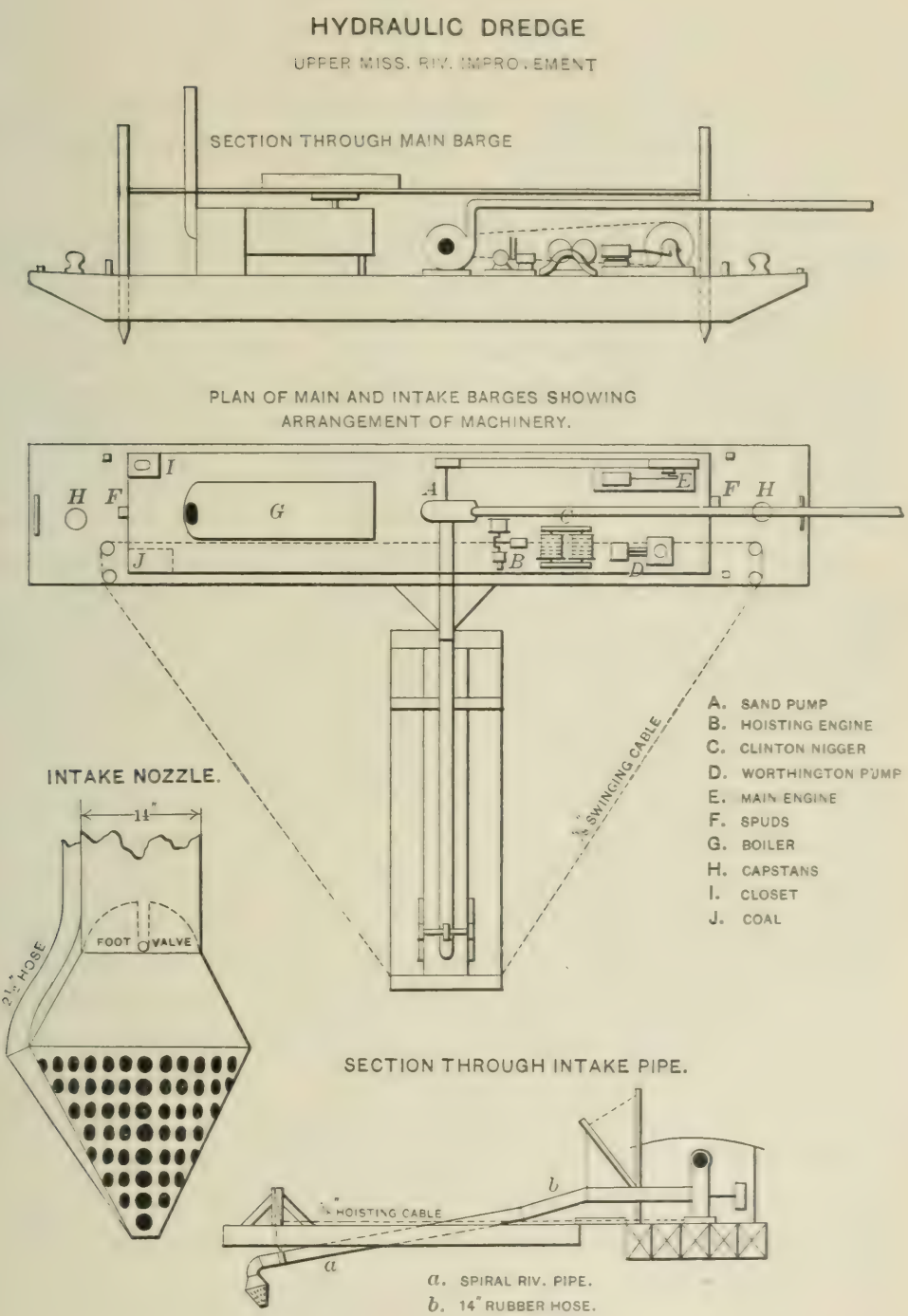

HYDRAULIC DREDGE
UPPER MISS. RIV. IMPROVEMENT
SECTION THROUGH MAIN BARGE
PLAN OF MAIN AND INTAKE BARGES SHOWING ARRANGEMENT OF MACHINERY.
A. SAND PUMP
B. HOISTING ENGINE
C. CLINTON NIGGER
D. WORTHINGTON PUMP
E. MAIN ENGINE
F. SPUDS
G. BOILER
H. CAPSTANS
I. CLOSET
J. COAL
INTAKE NOZZLE.
14"
FOOT VALVE
2½" HOSE
¾" SWINGING CABLE
SECTION THROUGH INTAKE PIPE.
¾" HOISTING CABLE
a. SPIRAL RIV. PIPE.
b. 14" RUBBER HOSE.

FIG. 6.

a week, and the channel remained open throughout the season. Part of the material was removed by dump-scows, but the greater part of it was dropped beside the cut, which was located parallel to the thread of the current.

This plant was also used at other places with good results. Major Ruffner, the engineer officer in charge, in the following words, recognized the utility of dredging :*

"The experience of the past season has shown that much assistance can be given to navigation by this kind of work, which, though temporary in a certain sense, is likely to be permanent for one season if not longer. * * * By the temporary use of dredges, at an expense so comparatively small as to be fully justifiable, I have no doubt a good navigable channel can be maintained during the low-water season on this stretch of the river, and if the navigation of the river is to be continued with success by steamboats, such temporary aid must be given until such time as the permanent channel improvement is completed."

In 1887 a contract was entered into for a hydraulic dredge and decked flat-boats to support 500 ft. of discharge pipe, to be used in the district above named. Experience had shown that a dipper dredge could do good work under favorable conditions, but for opening temporary channels through bars it was found to be too slow in its operations and hence too expensive. A dredge provided with a centrifugal pump for pumping sand and water from the channel and delivering the same through pipes far enough away to clear the channel was manifestly an improvement on the old method with bucket-dredges and dump-scows (see Fig. 6). This hydraulic dredge was equipped with a Van Wie No. 12-pump, with a 14-in. intake and a 12-in. discharge. The pump was run at the rate of 330 revolutions per minute and the material was lifted about 9 ft. above the surface of the water and discharged from 300 to 800 ft. from the pump. The intake nozzle was provided with a jet supplied by a Worthington pump, to aid in stirring up the material so that the suction could take hold of it.

The steam plant consisted of one boiler 22 ft. long, and having a 42-in. shell with ten 6-in. flues. The main engine was of the Wright and Adams pattern, 14-in. cylinder and 18-in. stroke. There was also a hoisting engine and a "nigger engine," each having 6 x 8-in. double cylinders.

The intake pipe is supported by a double pontoon, or two barges 4 by

* Report of Chief of Engineers, U. S. A., 1884, pp. 1561-1567.

50 ft., framed together with a 4-ft. space between them. The dredge end of this pontoon is pivoted to a triangular brace attached to the dredge, and the outer corners of the pontoon are connected with the dredge by means of wire cable-guys, passing through sheaves to the "nigger engine," which serve to swing the pontoon, with the suction, around in a semi-circle with a radius of 48 ft. A hoisting cable serves to raise and lower the suction between the two parts of the pontoon. The intake pipe has a 14-ft. length of rubber that connects the iron pipe of the suction with the iron pipe from the dredge to give the pipe sufficient flexibility. All engines are located so as to be handled by one man.

The machinery is mounted on a wooden hull 20 ft. wide, 100 ft. long and 4 ft. deep, and is housed in by a suitable cabin. There is a spud 12 ins. square at each end of the dredge which serves to hold it in the desired position. There are a number of small flat-boats 12 by 32 by 2 ft., each of which carries 45 ft. of discharge pipe. The several lengths of pipe are connected together with rubber hose. The flat-boats are coupled together by triangular wooden frames bolted to the decks, and so arranged as to couple with an iron pin midway between the boats. The discharge pipes are carried on a frame erected so as to give a slope of about 1 ft. in 100 ft.

This dredge is manipulated by means of two head lines anchored well up stream, and the dredge is moved and kept on line by swinging first on one spud and then on the other, alternately. The work can be done either with or against the current, each cut being about 90 ft. in width. The dredge is moved about 8 ft. at a time. A vacuum gauge at the suction side of the pump and a pressure gauge attached to the discharge side serve to show the operator whether the pipes are obstructed or whether the proper proportion of sand is being carried. This dredge delivers from 3 to 7% of sand through 600 ft. of discharge pipe at the rate of 70 to 100 cu. yds. per hour. It requires a crew of 7 men, costing $355 per month, to operate the dredge. The cost for 90 working days, including delays of all kinds, and the current repairs, was $2 928.71. During this time the dredge removed 42 900 cu. yds. of sand, at a cost of 6.8 cents per yard.

It is used not only to deepen channels, but also to fill in dikes and dams. This is quite an advantage where the amount of sediment carried by the stream is small.

This plant has been operated by Assistant Engineer J. Du Shane, and the above description is largely drawn from his report on the plant, which, together with other data, was obtained by the author from the U. S. Engineers located at Rock Island. The description here given applies to the dredge as remodeled in 1895.

This dredge evidently was regarded as fairly successful in that part of the river where first brought into use, as Major Ruffner, then in charge, says:*

"I believe that patience and some mechanical skill will make our machine able to dig a channel through any of our ordinary bars here, deep enough to induce the main flow of water through it in three days' work. * * * If this can be done there would be no difficulty in opening and maintaining a good navigable channel through this stretch of the river (between the Des Moines and Illinois Rivers) when the water is falling, by the use of two or three of these hydraulic dredges constantly operating and moving from the worst bar to the next one needing dredging."

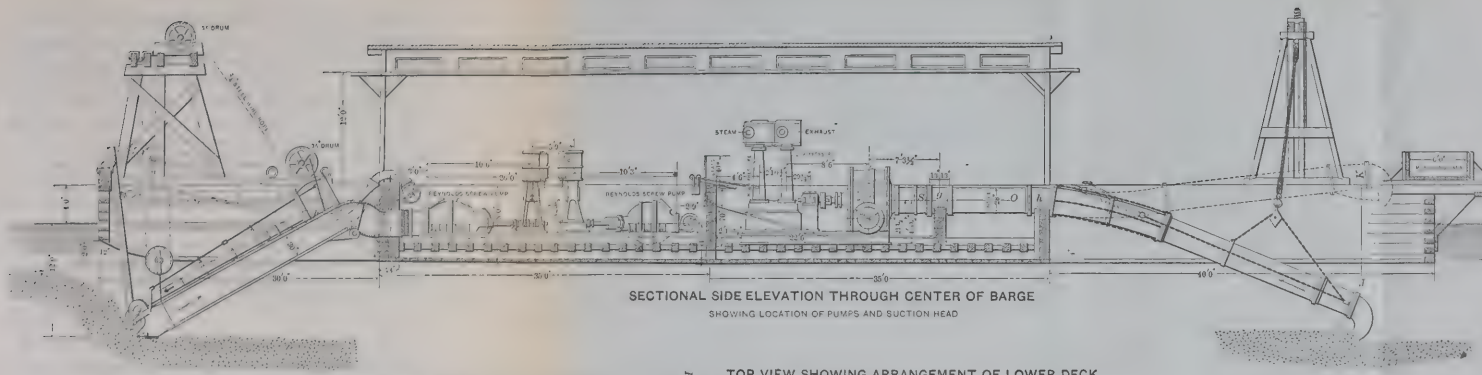
While this dredge was evidently a move in the right direction and was very useful in the upper river, its capacity was entirely inadequate to meet the requirements of the lower river. Here the dredging required at a single bar often exceeds 100 000 cu. yds., and the time permissible to remove this large amount of material is very short. To successfully accomplish this result, machines were required of far greater capacity than any heretofore used or known.

In November, 1891, the representatives of steamboat transportation lines below St. Louis appeared before the Mississippi River Commission and urged immediate relief for low-water navigation. They stated that if such relief depended on the completion of the permanent improvement work, by the time it came, there would be no traffic. In response to this appeal, the Commission appointed a committee of two:

"To study the subject of the construction and operation of such appliances for dredging as can be applied to the deepening of the Mississippi River over the bars in extreme low water;" * * * and "Prepare and report a project for the construction of a dredging boat of as large capacity as can be handled in the channel of the Mississippi River at low water with safety and convenience."

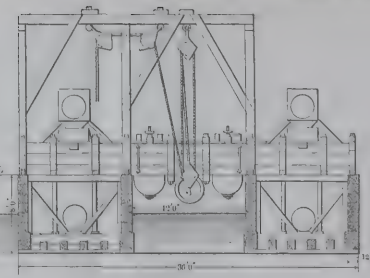
* Report of Chief of Engineers, U. S. A., 1887, p. 1614.

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XL, No. 838.
OCKERSON ON DREDGES AND DREDGING.



SECTIONAL SIDE ELEVATION THROUGH CENTER OF BARGE

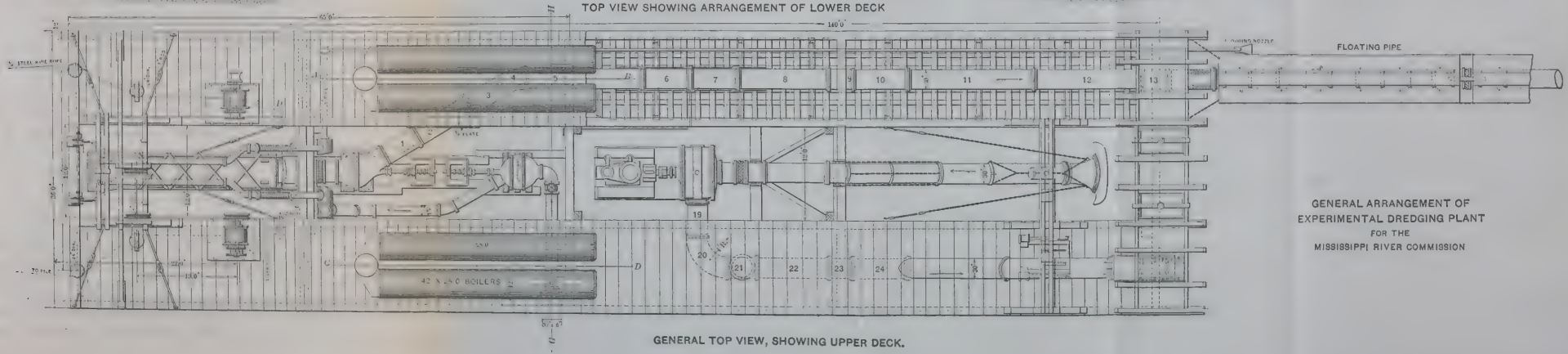
SHOWING LOCATION OF PUMPS AND SUCTION HEAD



SECTION ON LINES J.K.L.

SHOWING HOISTING ARRANGEMENT FOR SUCTION HEAD (EDWARDS PUMP)

TOP VIEW SHOWING ARRANGEMENT OF LOWER DECK



GENERAL TOP VIEW, SHOWING UPPER DECK.

GENERAL ARRANGEMENT OF
EXPERIMENTAL DREDGING PLANT
FOR THE
MISSISSIPPI RIVER COMMISSION

It should here be stated that the law under which the Commission were operating required them to prepare a plan to permanently locate and deepen the channel, and hence temporary expedients were naturally omitted from their plans up to this time.

Colonel Chas. R. Suter and Henry Flad, both Members of this Society, were designated to serve on this committee. After a thorough investigation of the matter, the committee concluded that among the various devices for temporary relief, dredging was the only means which held out a reasonable promise of success. The problem involved so many new features in the way of the enlargement and combination of devices heretofore used, that the construction of an experimental dredge was recommended. Drawings and specifications were prepared under the direction of this dredging committee, for an experimental dredge, and contracts for the construction of the various parts were let, at the close of 1892.

This experimental dredge was equipped with an Edwards centrifugal pump having a curved drag-suction at one end, and an Allis-Reynolds screw pump having a straight suction with jet agitators at the other end, the object being to make practical tests of the efficiency of the different devices. The pumps, engines and boilers were designed by their respective manufacturers. The Edwards pump runner was 6 ft. 4 ins. in diameter, and was operated by a compound, non-condensing, vertical engine with cylinders 15 ins. and 27 ins. in diameter, respectively, and with 20-in. stroke. The suction and discharge were each 30 ins. in diameter. The Allis-Reynolds pump was operated by a compound non-condensing vertical engine, with cylinders 8 and 16 ins. in diameter and with 12-in. stroke. The suction and discharge pipes were each 30 ins. in diameter. The jet pump was of the Allis-Reynolds pattern, with suction and discharge 15 ins. in diameter. This pump operated six jets $2\frac{1}{2}$ ins. in diameter under a head of about 20 ft. By this means the material was loosened and divided up so as to enter the suction of the sand pump thoroughly mixed with water.

The steam for the various pumps, etc., was supplied from four boilers 38 ins. in diameter and 28 ft. long, of the type in common use on Mississippi River steamers. Hoisting apparatus was also

provided for raising and lowering the suction and moving the dredge.

The above-described machinery was mounted on a wooden hull 130 ft. long, 36 ft. wide and 8 ft. in depth. The dredge in working order drew about 4 ft. of water. A good general idea of this dredge can be obtained by inspecting Plate XXIV.

The material pumped was discharged through a floating pipe line made up of sections of pipe 33 ft. long, connected by means of strong rubber joints and iron coupling bars. An air chamber on each side of the discharge pipe served to float it when loaded with 10% or less of sand. In order to operate the dredge, two hydraulic piles are set about 25 ft. apart, near the upper end of the bar to be excavated. A wire rope about 1 000 ft. long is attached to each of these and to the hauling drums on the dredge. If the wind does not deflect the dredge from its proper place, it is allowed to swing freely in the current with the discharge pipes trailing below it. If necessary, side piles are set, and by means of lines attached to them, the dredge can be hauled into such position as may be desired. In dredging, the boat is pulled ahead up stream at rates varying with the depth of the material taken out, the average movement being about 60 to 75 ft. per hour. After one cut is finished, the dredge is dropped down again to the lower end of the bar, the head piles are moved over about the width of the cut and another cut is made near by and parallel to the first. This process is repeated until a channel of the required width and depth has been secured.

There is perhaps no single feature which facilitates this method of dredging as much as the hydraulic mooring piles (Fig. 7). The ease with which they can be set and withdrawn, and their great holding power, make them really indispensable where dredging is done in strong currents. They are sunk 15 or 20 ft. deep in the sand, and the head lines are attached to a shackle down at the surface of the sand, and consequently there is little danger of bending. A pile sinker provided with suitable leads and hoist, pumps and boilers, is required with each dredge. The pile is open at the lower end for the full size of the pipe. On the side of the pipe near the upper end is an opening $2\frac{1}{2}$ ins. in diameter to which the pressure pipe from the pump is attached. The ease with which piles can be set and withdrawn is clearly set forth in Table No. 1, page 250.

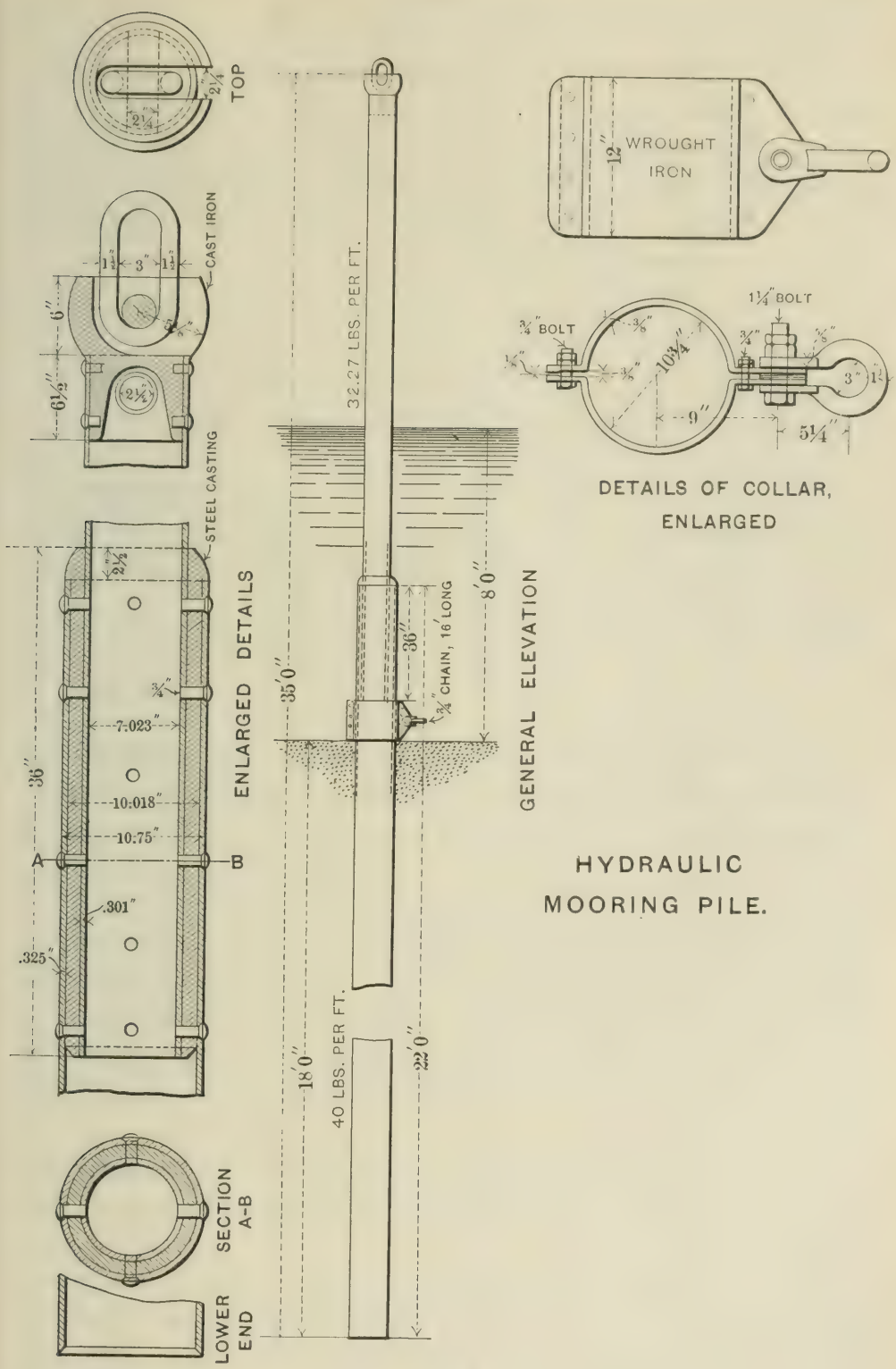

TOP
CAST IRON
STEEL CASTING
ENLARGED DETAILS
SECTION A-B
LOWER END
32.27 LBS. PER FT.
GENERAL ELEVATION
40 LBS. PER FT.
WROUGHT IRON
DETAILS OF COLLAR, ENLARGED
HYDRAULIC MOORING PILE.

FIG. 7.

TABLE No. 1.—Experiments in Sinking Hollow Wrought-Iron Piles in Sand.

In these experiments one 6-in. inside-packed, double-plunger pump was used. The length of the piles was 34½ ft.; outside diameter, 11 ins.; thickness, ¾ in. for the heavy piles, which weighed 3 250 lbs. each. The light piles were of the same length and outside diameter; thickness, ½ in.; weight, 2 370 lbs. Diameter of water-pressure inlet 2½ ins. The piles were open at the bottom.

Number of test.	Depth of water.	Depth of penetration.	Pressure required.	Time required to sink.	Pile left undisturbed.	Time required to raise.	Boiler pressure.	Kind of pile.	Remarks.
	Feet.	Feet.	Lbs.	M. S.	H. M.	M. S.	Lbs.		
1..	6.0	19.5	40	3 46	1 05	1 12	70	Light	Both hoist and pump used to draw.
2..	7.5	19.5	40	3 34	1 15	1 35	65	Heavy	
3..	7.0	19.0	35	4 53	1 30	1 20	60	Heavy	Hoist only used to draw pile.
4..	8.0	20.0	35	6 00	1 50	1 35	73	Light	
5..	8.5	19.0	30	6 00	0 35	1 13	70	Heavy	Both hoist and pump used.
6..	8.0	18.0	30	4 29	16 00	1 25	68	Heavy	
7..	12.0	19.0	30	3 23	16 00	1 26	74	Heavy	
8..	12.0	18.0	35	3 26	1 55	1 11	68	Heavy	Hoist only used to draw.
9..	11.0	19.0	35	4 28	0 55	1 05	63	Heavy	
10..	2.5	20.0	55	4 00	0 22	1 58	80	Heavy	
11..	3.0	19.0	55	8 30	0 15	1 51	70	Heavy	

The various parts of the experimental dredge were assembled and erected under the immediate supervision of Henry Flad, M. Am. Soc. C. E., in the fall of 1893. The tests were begun in November of that year and continued throughout the winter and the following spring. As a result of these tests the Allis-Reynolds pump at the bow of the boat was found to be deficient in engine power and the drag-suction at the stern of the boat could not be manipulated so as to give a regular and satisfactory supply of material. The Edwards pump gave good results and the jet agitator suction-head was found to be satisfactory. The jet pump required extensive alterations before the required results could be secured.

The buoyancy of the floating pipe line was too small and great care was required to prevent sinking the pipes by getting a trifle over 10% of sand in them.

Another unexpected difficulty developed in the use of these pipes. When swinging freely in the current with the lower end open and the pump running, the pipes would kink up at the joints. This was attributed to the reaction of the discharge against the water in which

the pipes were floating. After considerable experimenting with jets discharging in the air and in water, the conclusion was reached that the kinking was due to the centrifugal force of the flowing water acting at the flexible joints of the pipe. That is to say, when the pipe swings freely in the current it is never straight, and any bend met with by the water flowing in the pipes at a high velocity is naturally exaggerated. This difficulty was fully overcome by attaching a baffle plate to the last section, a short distance from the end of the pipe, to receive the impact of the discharge. By shifting this baffle plate so that the discharge will strike it at an angle, the pipe line can be deflected to the right or left.

The experiments had solved many of the difficulties, and attention was then turned to utilizing this dredge for practical work. The Allis-Reynolds pump and the drag suction were discarded. The Edwards pump was shifted and attached to the jet suction. In the spring of 1895 a cabin was built for the accommodation of the crew, and the first dredge for the elimination of obstructing sand bars on the lower Mississippi river was ready for actual work during the low-water season of 1895.

In the fall of 1894 the first attempt to aid navigation was made near Cape Girardeau, Mo., where there was a bar about 1 600 ft. long with from 3 to 4 ft. of water. Work began on October 19th, and by October 26th a channel 6 ft. deep had been made and this channel remained good throughout the season.

During the winter of 1894–5 a large number of tests were made to determine the capacity of the dredge. The immense amount of material constantly moving along the bottom of the river and the scouring effect of the current, made it impracticable to ascertain the amount moved, by measuring the material in place and then measuring the excavation. A measuring barge was therefore fitted up with suitable valves, to attach to the lower end of the discharge pipe where the discharge could be deflected into the barge for a known interval of time when the running conditions were about normal. This barge is 107½ ft. long., 24 ft. wide and 6¼ ft. deep. A special floor was laid in the bottom, and tight bulkheads were built near each end. The gunwales were stiffened with trusses in order to carry with safety the unusual load put in the barge. A 6-in. gate-valve was placed below the floor, next to the bottom planking at each corner of the loading

space, or spoil bin, of the barge. Pipes lead along the gunwales to each of the valve pipes, and are arranged so as to be connected with the pressure pumps of the pile sinkers. A well with removable sides is placed around each gate-valve so that when the barge is filled the water may be drawn from the top without disturbing the sand. There are two gauges placed on each side of the spoil bin and near the ends of the same. By means of simultaneous readings on these gauges the depth of material is determined. In making a test the barge is attached to the lower end of the pipe-line by means of a rubber pipe 6 ft. long and of the same diameter as the floating discharge pipe. This carries the discharge into the barge and to a point near the forward bulkhead of the spoil bin. Here a valve is provided that will deliver the discharge either into the river through an opening in the bottom of the barge or into the spoil bin, as may be desired. This valve (shown in Fig. 8) operates, both in opening and closing, by releasing heavy weights which cause the valve to revolve almost instantaneously.

Fig. 8 shows the opening to the spoil bin closed and the outlet through the bottom of the barge into the river, open. The brakes shown, hold the valve in any desired position. In setting the weights ready for a test, the weight *W*, which is shown at the bottom of the bin, is raised about 2 ft. above the button *O*, and supported by a rope attached to the framework above. It is also attached to the button *O* by a second rope. When ready to throw the valve down, so as to discharge into the spoil bin, the first rope is cut, which allows the weight *W* to drop about 2 ft., and the entire weight *W* is taken by the cable *K* which revolves the valve to its lower position, closing the outlet to the river and opening into the spoil bin. The brakes controlled by the lever *F* are put on to hold the valve in place, and the weight *W* is entirely released and drops to the bottom. The weight *A*, used for closing the valve to the spoil bin, is supported by the hook *N* some 2 ft. above the button *B* at the end of the cable *C* which passes through the weight, the opposite end passing around a pulley fixed to the valve shaft. When the spoil bin is full, the weight *A* is dropped from the hook *N* by raising the lever, and at the same instant the brakes are released and the valve moves back to its first position, throwing the discharge into the river. It is found necessary to start the valve with a jerk.

In making a test, the dredge pumps are started and the dredge is pulled ahead as in actual work, the barge-valve being set so as to discharge the spoil into the river. Sufficient water is let into the spoil bin through the gate-valves to cover the floor, and the gauges are read. This is done so as to be sure that none of the volume pumped in will go under the floor where it cannot be measured. When everything on the dredge is running smoothly and at about the normal rate, the dis-

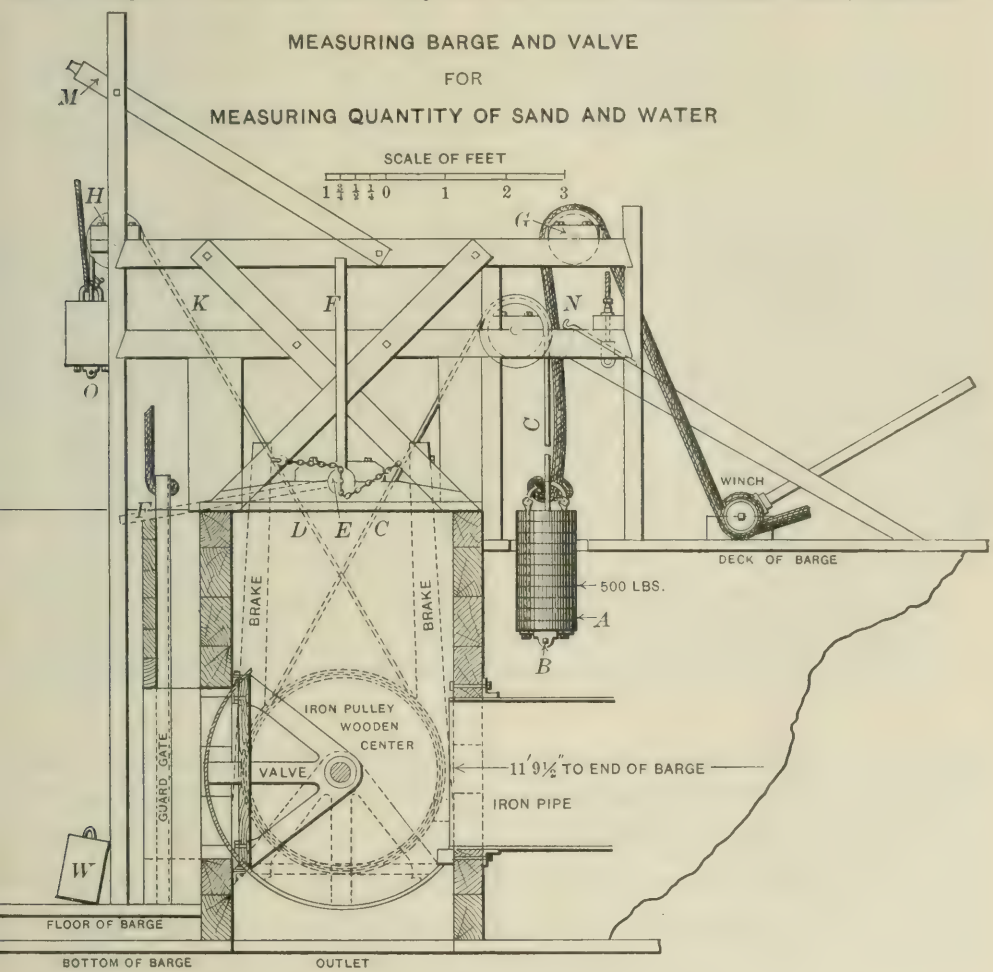


FIG. 8.

charge-valve into the spoil bin is thrown open and the timing hand of the stop-watch is started. In throwing the valve open to the spoil bin, the opening to the river is closed by the same valve. When all the material that the barge will safely hold has been discharged into the barge, the valve to the spoil bin is closed and opened to the river again. At the same instant the timing hand of the stop-watch is stopped. After a few minutes' time, necessary for the agitated water in

TABLE No. 2*a*.—Capacity and Efficiency Tests

Number of test.	Duration of test, in seconds.	Depth of Water at Bow.		Rate of movement ahead of dredge. Feet per second.	Edwards Pump.											Jet Pump, 6 Jets. 2½ ins. Diameter	
		Port.	Star.		Revolutions per minute.	Steam pressure.	I. H. P.			Head.						Delivery head. Feet of water.	Revolutions of jet pump.
							H. P. Cyl.	L. P. Cyl.	Total.	Suction head.		Mean delivery head. Feet of water.	Mean velocity head by barge measurement. Feet of water.	Total mean head. Feet of water.			
										Mean head. Ins. of mercury.	Mean head. Feet of water.						
7.....	150	6.5	7.3	.075	118	154	166	153	319	9.18	10.40	15.53	2.75	28.68	20.5	224
8.....	138	7.0	8.0	.071	118	153	165	154	319	9.32	10.56	15.80	1.90	28.26	20.3	224
9.....	122	5.5	6.5	.037	117	150	165	147	312	9.21	10.44	15.39	2.11	27.94	21.0	234
10.....	140	6.0	6.5	.045	116	143	154	137	291	8.65	9.80	15.38	2.11	27.29	20.8	230
11.....	155	6.5	7.3	.058	119	155	166	153	319	9.20	10.43	16.59	2.40	29.42	20.9	230
12.....	138	6.5	7.5	.037	118	150	164	155	319	8.98	10.18	15.23	2.23	27.64	20.1	228
13.....	123	8.2	8.3	.037	115	150	166	150	316	9.29	10.53	14.08	2.58	27.19	21.0	230
14.....	135	7.0	7.7	.037	118	150	163	151	314	8.90	10.09	15.90	2.02	28.01	19.8	226
15.....	131	8.2	8.0	.030	119	153	167	154	321	9.33	10.57	15.32	2.61	28.50	21.0	236
16.....	183	10.0	9.5	.037	123	146	159	147	306	7.11	8.06	22.13	1.48	31.67	20.5	228
17.....	194	5.5	5.0	.025	127	138	156	138	294	7.62	8.63	22.80	0.88	32.31	20.2	224
18.....	197	5.5	8.0	.030	130	140	159	140	299	6.42	7.27	22.64	0.96	30.87	20.8	230
19.....	174	5.0	5.0	.050	130	146	166	152	318	7.16	8.12	23.53	1.32	32.97	20.2	222
20.....	193	7.0	6.2	.021	125	130	145	129	274	6.44	7.30	25.52	1.24	34.06	19.8	216
21.....	184	3.5	4.0	.050	129	143	167	151	318	7.17	8.13	23.36	1.39	32.88	21.0	230
22.....	180	5.0	5.0	.050	120	140	155	145	300	7.68	8.70	20.73	1.80	31.23	20.0	226

The first nine tests were made with 604.5 ft. of pipe between the pump and the valve
Diameter of discharge pipe, 30 ins. Distance from suction head to pump, 101 ft.
The area of spoil bin in test barge, 2 544.01 sq. ft.
The average capacity of sand per hour with the short discharge pipe was 679 cu. yds.,
The sand pumped per hour per indicated horse-power with the short discharge was

the spoil bin to settle down, the gauges are read, the pumps on the pile sinkers are started, the gate-valves are opened and the water is forced out, leaving the sand to be measured later. This is done by measuring the depth on eleven cross-sections at eleven points on each section, all equally spaced. The average depth multiplied by the known area gives the volume of sand. The differences of the gauge readings before and after filling give the depth of material pumped in. This multiplied by the area gives the total volume of sand and water. From this, and the time required to pump such volume, the various functions of capacity per hour, velocity per second, efficiency, etc., are determined. The material enters the barge with such force that the sand is not distributed uniformly over the bottom, but piles up in ridges on one side or the other.

Sometimes the sand is measured in a box 1 ft. deep, 5 ft. wide,

OF UNITED STATES DREDGE *ALPHA*. 1894–95.

Measuring Barge.									Sand.		Efficiency.		
Mean depth of water in barge before test.	Mean depth of water in barge at end of test.	Mean depth of water pumped.	Cubic feet of material pumped.	Cubic feet per second.	Velocity per second. Feet.	Cubic feet of sand pumped.	Percentage of sand.	Capacity of pump, in cubic yards per hour.	Weight of 1 cu. ft., dry.	Percentage of voids.	Work done in foot-pounds per second as measured by barge.	Work done in foot-pounds per second as measured by indicator cards.	Percentage.
.626	4.478	3.852	9 799.52	65.33	13.31	552.0	5.63	491	106	35	117 106	175 450	66.75
.707	3.663	2.956	7 520.09	54.49	11.10	936.5	12.45	905	104	33⅓	96 244	175 450	54.85
.575	3.340	2.765	7 034.18	57.66	11.74	766.5	10.89	838	104	38	100 688	171 600	58.67
.438	3.610	3.172	8 069.60	57.64	11.74	614.0	7.61	585	108	36⅔	98 312	160 050	61.42
.406	4.120	3.714	9 448.45	60.96	12.41	970.0	10.27	835	104	37⅓	112 062	175 450	63.87
.505	3.713	3.208	8 161.18	59.14	12.04	520.0	6.37	503	109	36⅔	102 187	175 450	58.24
.398	3.450	3.052	7 764.31	63.12	12.85	715.0	9.21	775	109	33⅓	107 250	173 800	61.71
.473	3.465	2.992	7 611.67	56.38	11.48	660.0	8.67	652	106	36⅔	98 687	172 700	57.14
.448	3.720	3.272	8 324.00	63.54	12.94	516 3	6.20	526	106	38⅓	113 187	176 550	64.11
.562	4.018	3.456	8 792.09	48.04	9.78	460.0	5.23	335	100	41⅔	95 062	168 300	56.48
.438	3.260	2.822	7 179.19	37.01	7.53	530.0	7.38	364	108	38⅔	74 750	161 700	46.23
.461	3.448	2.987	7 598.95	38.57	7.85	925.0	12.17	626	106	36⅔	74 437	164 450	45.27
.557	3.647	3.090	7 860.99	45.18	9.20	716.0	9.11	549	108	37⅓	93 125	174 900	53.25
.530	3.857	3.327	8 463.92	43.85	8.94	586.6	6.93	405	111	34⅓	93 375	150 700	62.62
.616	3.975	3.359	8 545.32	46.44	9.46	663.0	7.75	446	107	40	95 438	174 900	54.57
.643	4.392	3.749	9 537.49	52.98	10.79	849.0	8.90	629	103	41⅔	103 438	165 000	62.69

in the test barge. The other seven tests were made with 1 059.5 ft. of pipe.
Diameter of suction pipe, 30 ins. The suction was lowered 12 ft. below the water surface.

and with the long pipe, 479 cu. yds.
2.16 cu. yds., and with the long line, 1.59 cu. yds.

and 10 ft. long. The floor of the barge is cleared off so as to give room for the box. The sand is then shoveled into it without packing, and the top is smoothed off with a straight edge. This method gives about 22% more volume than when the sand is measured in place, but as it necessitates handling the material twice, it has been abandoned.

The results of twenty-two tests of the dredge *Alpha*, with the Edwards pump, are given in Table No. 2*a*.*

A new pump, similar to that on the *Delta*, with a 66-in. runner, 33 ins. wide, was put in the *Alpha* in July, 1897. Barge tests with this pump were made in June, 1898. the results of which are shown in Table No. 2*b*.

The time spent in experimenting with the first dredge was by no means wasted. There were many difficult problems of construction

* Report of C. W. Sturtevant, in the report of Chief of Engineers. U. S. A.. 1895. p. 3795.

TABLE No. 24.—Capacity and Efficiency Tests

Number of test.	Duration of tests, in seconds.	*Conditions.				Main Engines.					Main Pump.				Jet Pump.	
		Length of discharge pipe from stern of dredge to valve in measuring barge, in feet.	Depth of suction, in feet.	Depth of water, in feet.	Forward movement of dredge, in feet, per minute.	Steam pressure.	Revolutions per minute.	I. H.-P.			Head.—Feet of Water.				Revolutions per minute.	Head.
								H. P. Cyl.	L. P. Cyl.	Total.	Suction.	Delivery.	Velocity.	Total.		Delivery, feet of water.
1..	69.5	525	12	....	....	160	156	217.3	250.0	467.3	9.6	25.8	3.1	38.5	....	.....
2..	95	525	12	6.9	6.9	165	152	217.3	236.0	453.3	13.6	30.4	3.0	47.0	235	20.7
3..	93.5	1 057	12	7.5	2.2	165	159	186.1	151.6	337.7	7.4	35.0	1.4	43.8	230	20.0
4..	101.5	1 057	12	5.7	2.9	125	148	170.6	187.0	357.6	10.2	35.0	1.2	46.4	228	19.8
5..	101.5	1 057	10	6.2	6.5	156	155	198.0	199.2	397.2	10.2	32.7	2.0	44.9	228	19.8
6..	102	1 057	12	....	....	155	155	207.5	217.3	424.8	7.4	24.6	2.4	34.4	....	.....
7..	101.5	1 057	12	5.6	5.4	160	157	183.1	178.7	361.8	10.2	53.4	0.8	64.4	228	19.7
8..	120	1 057	12	4.6	3.5	158	162	164.0	165.9	329.9	9.1	38.5	0.2	47.8	230	20.0
9..	109	1 057	10	5.3	6.3	155	152	196.2	194.4	390.6	9.1	†28.1	1.5	38.7	233	20.5
10..	63	525	12	6.7	6.8	160	152	213.1	262.8	475.9	14.2	30.4	3.1	47.7	233	20.5

* Suction pipe, 30 ins. diameter; discharge pipe, 30 ins. diameter. Distance from

† Delivery head observed 12 minutes before test.

and manipulation solved, and the efficacy of dredging at all in a stream which moves vast quantities of material on its own account was tested to some extent. The results of all these experiments pointed clearly to dredging as the most promising method of temporarily improving low-water navigation that had so far been suggested or tried. The time had now come to provide a sufficient number of dredges to clear the obstructing reefs in so short a period of time that the interruption to navigation would be very slight. Dredging, however, had not yet been authorized specifically by Congress, and the amount of money available for the construction of dredges was rather limited.

With a view to covering the ground still more fully, and to get the benefit of the best dredging experience available, a circular was sent out in August, 1894, to engineers, contractors and builders of dredges, inviting "Informal plans and suggestions as to the methods and machinery best adapted to the work." The capacity of the required dredge was to be 1 600 cu. yds. of sand per hour. Several replies were received, and three firms were requested to prepare

OF NEW PUMPS ON THE DREDGE *ALPHA*, JUNE, 1898.

Measuring Barge. Area of water = 2 558.7 sq. ft. Area of sand = 2 532.5 sq. ft.								Efficiency.					
								Foot pounds of work done per second.					
Mean depth of material pumped, in feet.	Quantity of material pumped, in cubic feet.	Quantity of material, in cubic feet, per second.	Velocity of discharge, in feet, per second.	Mean depth of sand, in feet.	Quantity of sand, in cubic feet.	Percentage of sand.	Sand pumped per hour, in cubic yards.	Measured by material pumped into measuring barge.	Measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, in cubic yards, per I. H.-P.	Kind of material.	Remarks.
1.90	4 861.53	69.95	14.2	.....	.....	....	.....	168 317	257 015	65	....	.....	Water test.
2.53	6 473.51	68.14	13.9	0.454	1 149.8	18	1 614	200 161	249 315	80	3.56	Med. fine sd.	
1.73	4 426.55	47.34	9.6	0.244	617.9	14	881	129 593	185 735	70	2.61		
1.68	4 298.62	42.35	8.6	0.0731	185.1	4	243	122 815	196 680	62	0.68	Fine sand	
2.23	5 705.90	56.22	11.4	0.2632	666.5	12	876	157 767	218 460	72	2.21	Med. fine sd.	
2.42	6 192.05	60.71	12.4	.....	.....	....	.....	130 526	233 640	56	....	.....	Water test.
1.42	3 633.35	35.80	7.2	0.3531	894.2	25	1 175	144 095	198 990	72	3.25	Med. fine sd.	
0.90	2 302.83	19.19	3.9	0.1883	476.9	21	530	57 330	192 445	30	1.61		
2.03	5 194.16	47.65	9.7	0.2931	742.3	14	908	†115 253	214 831	†54	2.32		
1.70	4 349.79	69·04	14.1	0.2800	709.1	16	1 501	205 825	261 745	79	3.15	.....	

pump to end of suction, 66 ft.; from pump to stern of dredge, 74 ft.

detailed drawings and specifications of the plant they proposed to furnish. These were submitted in December, 1894, and a contract was let to the American Hydraulic Dredging Co. of Chicago, Ill., for the construction of the dredge *Beta*, at a cost of $172 775. The contract provided that if the dredge fell short of the required capacity, the deduction from the price should be proportional to this difference; and if the capacity exceeded 1 600 cu. yds. per hour, then the price should be increased proportional to this excess up to 50 per cent. The dredge was completed and ready for testing in January, 1896.

DREDGE *BETA*.

This dredge has two independent dredging machines complete from suction to end of discharge pipes. The sand pumps are of the centrifugal pattern with runners 7 ft. in diameter and with 8 arms in each runner. The suction for each pump is 33¾ ins. in diameter, and is split near the pump so that the water enters the casing from both sides. The discharge for each pump is from a single 33-in. pipe leaving the pump chamber from the top. The suction for each pump divides near the forward end of the hull into three suctions, each 19½

ins. in diameter. A cast-iron elbow is riveted to the hull at the bow for each of the three branches, which are provided with elbows of hammered copper $\frac{3}{8}$ in. thick, and these work in the iron elbows and form a radial, telescopic joint. The axis of rotation is the center of a 7-in. shaft supported on brackets at the bow of the boat. The elbows are concentric with this shaft. These three suctions are framed together and can be raised or lowered, as one piece, through a depth of 20 ft.

Each of these suctions is provided with a vertical revolving cutter, 5 ft. in diameter and 5 ft. long, with 12 nickel-steel blades on each cutter. These serve to loosen up the material so that it will readily enter the suction. They are spaced 6 ft. apart from center to center. There are six of these revolving cutters for the two pumps. They are driven, through a system of spur gearing, by a cross-compound, non-condensing engine with $14\frac{1}{2}$ and 29-in. cylinders and an 18-in. stroke. The engines make about eight revolutions to one revolution of the cutters. The cylinders are some 20 ft. apart and are connected by a 5-in. pipe.

To support a portion of the weight of the cutter machinery, a pontoon is provided for each set of suctions. This pontoon is made of steel plates, and is built around the various pipes and frames. Each pontoon has a displacement of 1 000 cu. ft.

The sand pump shaft is of forged steel, 10 ins. in diameter, and 12 ins. in diameter where it passes through the pump runner. Stuffing-boxes are provided where the shaft passes through the pump casing. Each pump is operated by a direct-connected, triple-expansion, vertical, inverted, four-cylinder, tandem engine provided with jet condensers. The cylinders are $20\frac{1}{2}$, 33, 38 and 38 ins. in diameter, respectively, and the stroke is 24 ins. with the cranks set at 180 degrees. These engines are run at a speed of about 130 revolutions per minute, the indicated horse-power of each being about 1 250, with boiler pressure of 175 lbs. Each engine is provided with a Worthington duplex air pump and jet condenser, with cylinders 14 and 19 ins. and 15-in. stroke. They are also arranged to exhaust without condensing. The steam is supplied by four Heine boilers so arranged that they can be used together or independently, as may be desired. Each boiler has two shells 36 ins. in diameter and $18\frac{1}{2}$ ft. long. The water legs are connected by 171 tubes, $3\frac{1}{2}$ ins. in diameter and $15\frac{3}{4}$ ft. long. There is one smokestack to each pair of boilers, each stack

being 64 ins. in diameter and 57 ft. high above the breeching, or 73 ft. above the water-line. The main steam pipe between the boilers and engines is 12 ins. in diameter. The usual steam gauges, blow-off valves, etc., are provided.

The water supply for the boilers and journal bearings is filtered by means of two filters, each 11 ft. long and 6 ft. in diameter. The filter bed is supported on a perforated cast-iron bed fixed near the bottom of each filter. There are three layers of brass wire-cloth with 10, 60 and 10 meshes per inch, respectively. These support a bed of crushed quartz made up of layers of different degrees of fineness and about 2 ft. thick. The sediment which gathers on the filter bed is stirred up by two sets of arms which can be revolved by means of gearing, and the clear water from one filter is used to wash the other. The filters are supplied by a duplex pump with 8-in. suction and 7-in. discharge, and the water is carried through an alum coagulant to the tops of the filters under a pressure of about 60 lbs. The filtered water is drawn from the bottom of the filters by the boiler feed-pumps, and passes through a vertical pressure heater provided with one hundred 2-in. corrugated copper tubes $73\frac{3}{4}$ ins. long. The exhaust steam from the cutter, the winding engines and the pumps passes through this heater.

The winding engines are placed near the bow of the boat. There are six drums operated by a double-cylinder, link-motion, reversing engine with $12\frac{1}{4}$-in. cylinders and 15-in. stroke. Four of the drums are 24 ins. in diameter and 40 ins. long. Two of these drums are used for side warping, and are geared 25 to 1. The other two are used for pulling the dredge ahead, and are geared 30 to 1; 1 200 ft. of $\frac{7}{8}$-in. wire rope is used for this purpose. The two end drums are 30 ins. in diameter and 28 ins. long. They are used for raising and lowering the suctions.

At the stern of the boat is a steam capstan and a spud hoist. There are two vertical anchor-spuds and one push-spud at the stern of the boat. The anchor-spuds are of oak, 24 ins. square and 40 ft. long. These were intended mainly as pivots on which to swing the boat, so that a wide cut could be made by moving the suction on the arc of a circle. This method is not well adapted to streams with a strong current. These spuds and the push-spud are raised or lowered by means of a 3-drum friction hoist moved by a double 8 x 10-

in. engine. The push-spud is an iron beam 28 ft. long, with a steel shoe at the lower end. It can be raised up into a recess in the hull when not in use. The inner end of the beam is attached to the piston rod of a hydraulic cylinder 7 ft. long and 15 ins. in diameter. The piston rod passes through the hull plating in a stuffing-box. This spud has never been used.

The machinery is mounted on a steel hull 172 ft. long and 40 ft. wide. It is 7 ft. 2 ins. deep in the waist, and for 36 ft. from the forward end and 16 ft. from the after end it is 10 ft. 10 ins. deep. These deep portions are decked over, as are also the guards, for the entire length. The plating on the sides of the hull is $\frac{3}{8}$ in. thick; the other plating outside is $\frac{5}{16}$ in. thick. The bow is double-plated on the head and rake. The cross floors, spaced 2 ft. between centers, are 12-in. I-beams, weighing 32 lbs. to the foot. On these, at intervals of $3\frac{1}{2}$ ft., are longitudinal stringers of the same form and height, weighing 40 lbs. to the foot. These floor frames are decked over with $\frac{1}{4}$-in. plating and extend to within 6 ft. of the bow. The space between this plating and the bottom is divided into ten water-tight compartments. The hull is further divided into seven water-tight compartments above the water bottom by means of five cross bulkheads and one short longitudinal bulkhead.

The bases of the pumps, engines and the boilers are set on the water bottom. Two coal bunkers are provided, with a capacity of about 30 tons each. The dredge is fitted with a traveling crane of ample capacity to handle all parts of the machinery.

No quarters are provided for the crew of this dredge. The cabin is only sufficient to protect the machinery. The boat is equipped with an electric light plant and a full complement of shop tools, such as lathe, drill press, etc.

The requirements of the contract fixed the limit of draft at $4\frac{1}{2}$ ft., and the width at 40 ft. When the dredge was completed, the draft was found to be about $6\frac{1}{2}$ ft.

The weights of the various parts are about as follows: Hull, 633 340 lbs.; main engines and pumps, 183 576 lbs.; boilers, 405 880 lbs.; cutter engine, 27 460 lbs.; forward hoist, 51 500 lbs.; spud hoist, 20 450 lbs.; ladders, 56 800 lbs. The total displacement when running, with suction and discharge pipes, boilers, filters, heaters, etc., full of water, and a supply of coal on board, is about 1 175 tons.

FIG 1.

FIG. 2.

The discharge consists of two independent pipe lines, each 1 000 ft. long. A deflecting baffle plate is provided for the end of each line. Eight of the sections of each line are 100 ft. long, and four sections are 50 ft. long. When the pipes are empty, the pontoons draw about 14 ins. of water. The pontoons are made of $\frac{3}{16}$-in. tank steel, and are fitted with water-tight bulkheads about 16 ft. apart. In section they are the shape of a semi-cylinder, with an outer radius of $30\frac{3}{4}$ ins., and an inner radius of $16\frac{3}{4}$ ins. The discharge pipe lies in the float thus formed, and is made of $\frac{1}{4}$-in. steel. The different lengths are coupled about as described in connection with the *Alpha*.

Quite a number of defects were developed in the use of this dredge, many of which were remedied. At first the cutters all worked in the same direction, and the result was that the dredge was pulled sidewise out of the cut. This was remedied by changing one set of cutters to work in the opposite direction. The breaks in the gearing of the cutter machinery were frequent, although the spur gears were made of nickel steel, and the pinions of phosphor bronze. The 6-drum hoist also gave considerable trouble. The cutters were found to be very much longer than necessary, and their operation drew heavily on the boiler capacity. The unexpected draft developed difficulties in coaling, and in maneuvering the dredge over the shoal bars.

The preliminary tests of the dredging machinery began about the middle of January, 1896, and continued until March 1st. Of the four hundred working hours embraced in this period, two hundred and fifty-five hours were used in repairs and changes and seventy-four hours in actual pumping. The capacity tests were begun on March 10th, and continued for about a month. There were ten tests made in ordinary river sand, one gravel test and one water test. The tests were made in the test barge described in the preceding pages. Two valves were provided, so that both pipe lines discharged into the barge at the same time, if desired.

In the gravel test the face of the bank dredged was about 7 ft. high, and only about 1 ft. of it was gravel. In this test one valve was first opened and the barge filled. The water was then syphoned out, and the other valve was opened and the barge again filled. The gauge readings and time of each were, of course, noted. Tests 9 and 10 were made with only one valve open, the other pipe line discharging in the river.

TABLE No. 3.—Capacity and Efficiency

Number of test.	Duration of tests.	Depth of water.	Depth of suction.	Forward movement per minute.	Pumps' Revolutions per minute.		Average boiler pressure.	I. H.-P.			Suction head.		Delivery head.		Difference between elevation of gauges.	Velocity head.	Total head.	
											In feet of water.							
					Starboard.	Port.		Starboard engine.	Port engine.	Total.	Starboard pump.	Port pump.	Starboard pump.	Port pump.			Starboard pump.	Port pump.
	Secs.	Ft.	Ft.	Ft.											Ft.			
1	35	8.3	14.6	7.5	150	149	177	1 280	.....	2 561	19.3	19.8	22.7	25.2	+8.4	6.4	57.1	60.
2	68⅝	11.5	18.5	3.1	140	157	169	.....	1 296	2 593	15.9	18.2	30.6	34.5	.....	2.3	57.1	63.3
3	$37\frac{35}{100}$	10.0	18.5	2.6	161	157	161	1 293	.....	2 586	14.7	14.7	36.3	39.7	.....	2.1	61.5	62.9
4	64⅞	9.8	18.3	3.6	151	148	178	.....	1 223	2 447	14.7	17.0	35.1	26.1	.....	2.9	61.1	54.4
5	26⅛	9.2	17.5	5.0	153	154	176	1 196	.....	2 392	20.4	18.7	29.4	37.4	.....	2.8	61.0	67.3
6	55¾	9.5	17.6	4.0	144	151	173	.....	1 300	2 609	18.7	18.1	25.5	32.8	.....	3.0	55.6	62.3
7	$53\frac{85}{100}$	9.5	17.5	7.5	146	148	172	1 105	.....	2 211	17.0	19.0	31.7	27.6	.....	3.3	60.4	58.0
8	$50\frac{31}{40}$	10.1	18.4	10.0	147	148	174	.....	1 247	2 495	20.0	21.5	28.3	29.2	.....	2.8	59.9	61.9
9	102¾	9.5	18.7	8.6	141	143	176	1 170	.....	2 340	19.3	21.0	23.8	24.9	.....	2.5	54.5	57.3
10	$95\frac{8}{10}$	9.5	18.8	8.2	142	144	174	.....	1 199	2 398	19.3	20.4	24.9	24.7	.....	3.7	56.3	57.2
	119	....	....	....	149	152	170	1 187	.....	2 374	18.1	18.9	21.5	21.5	.....	4.0	52.1	52.8
	$78\frac{1}{10}$	4.5	11.5	3.7	146	157	163	.....	1 161	2 322	17.6	13.6	22.7	35.0	.....	3.2	51.9	60.2
	.....	....	....	....	....	....	....	.....	.....	.....	....	....	....	....	.....	....	....	....

Length of port suction from intake of pump = 85.9 ft. Length of port discharge pipe
Length of starboard suction from intake of pump = 100.3 ft. Length of starboard
Diameter of main suction = 33¾ ins. Diameter of discharge pipes = 33 ins. Area of
Revolutions of cutters per minute = 19. Average capacity of dredge in cubic yards

In all the tests the dredge was run for a considerable time in all respects as if doing actual dredging work, and the valves were opened at such times as the pumps were seen to be working under about normal conditions. In the mean time steam gauges were read, revolutions of pumps, cutter engines and winding drums were counted, and indicator cards were taken at intervals of about 5 minutes.

The results of these tests are shown in Table No. 3.*

The general conditions as to length of time required for efficiency tests and number of capacity tests were the same as described in connection with the *Gamma*.

The tests show an average capacity of 4 920 cu. yds. per hour, and, therefore, according to the terms of the contract, a bonus of $86 387.50 was earned by the contractor.

* Report of Assistant Engineer Wm. Gerig, in report of Chief of Engineers, U. S. A., for 1896, page 3643.

TESTS OF UNITED STATES DREDGE *BETA*.

Number of test.	Measuring Barge. Mean velocity. Feet per second.	Total material pumped per minute.	Amount of sand pumped per minute.	Percentage of sand.	Quantity of sand pumped per hour.	Depth of material pumped into barge.	Percentage of voids in wet sand.	Weight per cubic foot of dry sand.	Horse power developed as measured by material in barge.	Efficiency per centum.	Sand pumped per hour per indicated horse-power.	Remarks.
	Ft.	Cu. ft.	Cu. ft.		Cu. yds.	Ft.		Lbs.			Cu. yds.	
1	20.8	14 812	391	2.6	868	3.41	35.2	96.9	1 685	65.8	0.29	
2	11.9	8 479	1 281	15.1	2 847	3.82	32.6	98.1	1 098	42.3	1.10	
3	11.6	8 268	2 859	34.6	6 354	2.03	34.2	97.5	1 298	50.3	2.46	
4	13.8	9 809	2 245	22.8	4 988	4.18	33.7	98.8	1 298	53.0	2.04	
5	13.5	9 608	3 509	36.4	7 798	1.65	34.8	98.1	1 558	65.2	3.26	
6	14.0	9 956	2 183	21.9	4 850	3.77	35.1	97.8	1 335	51.2	1.86	Pumping sand and water.
7	14.0	10 000	1 743	17.4	3 872	3.54	32.5	99.4	1 300	58.8	1.75	
8	13.5	9 635	2 798	29.0	6 217	3.20	33.3	97.5	1 399	56.1	2.49	
9	13.9	9 870	2 847	28.9	6 327	3.33	34.6	97.5	1 318	55.3	2.70	One valve open.
10	14.9	10 639	2 287	21.5	5 082	3.35	34.8	97.5	1 367	57.0	2.12	One valve open.
	16.5	11 430	.....	....	.....	4.51	....	.....	1 134	47.7	....	Water test.
	14.4	10 505	499	4.7	.....	1.83	....	.....	1 165	50.2	0.48	Gravel test.
	14.1	10 107	2 214	23.0	4 920	3.22	....	.....	1 365	55.5	2.00	Averages of first ten tests.

from pump to valve in measuring barge = 1 161.7 ft.
discharge pipe from pump to valve in measuring barge = 1 147.7 ft.
measuring barge = 2 535.3 sq. ft.
per hour by box measurements = 4 920, or about 4 330 cu. yds. measured in place.

After completing the required tests, sundry repairs and alterations were made, to put the dredge in condition for actual work.

During the low-water seasons of 1896 and 1897 the enormous capacity of the *Beta* was fully demonstrated. An account of the character and cost of this work will be given later.

Before the dredge is again put into service for the low-water season of 1898, numerous changes will be made. The hull will be widened to 58 ft. and lengthened, so as to reduce the draft to 4½ ft. About $41 000 has been retained from the contract price to do this work, as the specifications required that the draft should not exceed 4½ ft. A cabin for the accommodation of the crew will be erected; the 6-drum hoist will be replaced by four detached single-drum hoists; the cutter engines, cutters and other parts belonging to the present suction will be replaced by a jet suction and pumps. This is done because it has been

found in actual practice that mechanical agitators are unnecessary in the sands of the Mississippi River. The wear and tear of the cutter machinery is a very large item of expense, to say nothing of the power required. The jet agitators are quite efficient, are much more economical in point of repairs and power required, and are more simple in manipulation. The push-spud and one anchor-spud will be discarded. The other anchor-spud will be moved to the bow of the boat. The spud-hoist will be discarded. The forward battery of boilers will be turned end for end in order to properly distribute the weights under the new arrangements, and to improve the facilities for coaling. Much of this work would probably be deferred were it not that the boat must be put into dry dock to make the hull repairs, and this opportunity is seized for making the changes outlined.

A general view of the *Beta*, as she will appear after remodeling, is shown in Fig. 9.

Dredge *Gamma*.

The Act of Congress of June 3d, 1896, was the first to formally recognize and require the use of dredge boats and other devices as an adjunct to the permanent improvement of the lower Mississippi River. The Act referred to requires that as much of the money appropriated as may be necessary:

"Shall be expended in the construction of suitable dredge boats and other devices and appliances, and in the maintenance and operation of the same, with a view to ultimately obtaining and maintaining a navigable channel from Cairo down, not less than 250 ft. in width and 9 ft. in depth, at all periods of the year, except when navigation is closed by ice."

This formal recognition of the necessity of temporary relief for low-water navigation resulted in the early construction of four more dredges.

The contract for the *Gamma* was let in July, 1896, to the Bucyrus Steam Shovel and Dredge Co., at South Milwaukee, Wis. The capacity required was 800 cu. yds. of ordinary river sand per hour, delivered through 1 000 ft. of pipe with a single centrifugal pump. The price paid was $85 530.60, and the time fixed for completion was nine months. This dredge was used in actual work throughout the low-water season of 1897, and was found to be very satisfactory, both in capacity and economy.

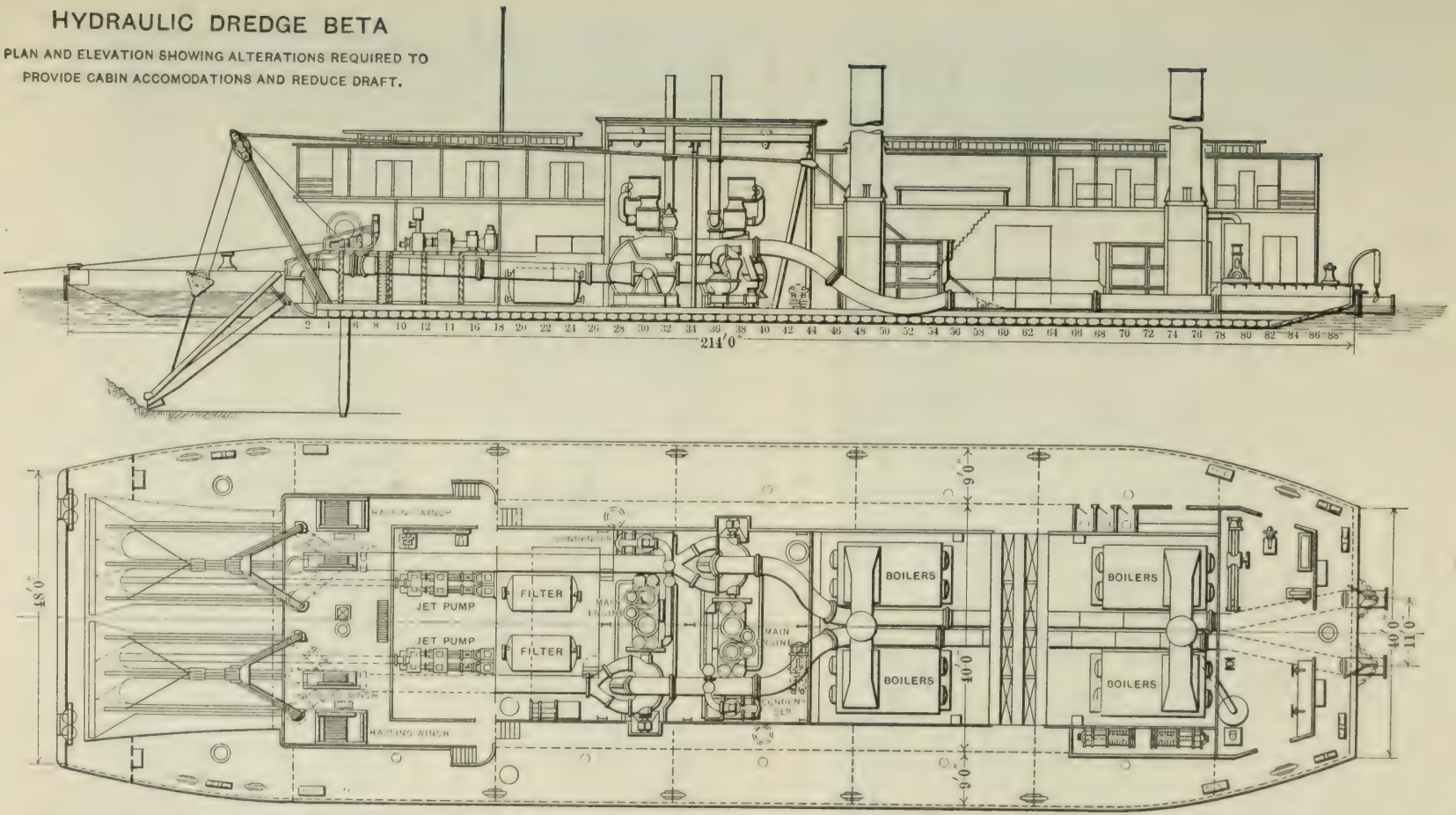

HYDRAULIC DREDGE BETA
PLAN AND ELEVATION SHOWING ALTERATIONS REQUIRED TO PROVIDE CABIN ACCOMODATIONS AND REDUCE DRAFT.
214'0"
JET PUMP
JET PUMP
FILTER
FILTER
BOILERS
BOILERS
BOILERS
BOILERS
HAULING WINCH
HAULING WINCH
MAIN ENGINE
MAIN ENGINE
48'0"
10'0"
10'0"
10'0"
11'0"

FIG. 9.

The hull of the *Gamma* is of steel, and is 138 ft. long over all, 38 ft. wide and 8 ft. deep. There is a well at the bow 32½ ft. long, in which the suction is placed.

The framing of the hull consists of 12-in. channels, stiffened with I-beam keelsons. The framing is shown in Fig. 1, Plate XXV. Cross bulkheads divide the hull into two compartments, one for the boiler, and the other for the engines. These are surrounded by water-tight compartments and are decked over.

The main dredging pump is located in the forward part of the engine room. It is a centrifugal pump, with a 24-in. suction inlet on each side and a 34-in. discharge at the bottom. The runner is 5 ft. 9 ins. in diameter, and has four blades. It revolves at a speed of about 150 revolutions per minute. The pump casing is split horizontally through the axis, so that the upper half can be removed in making repairs. The casing is of cast iron, from 1¾ to 2½ ins. thick. The shaft passes through the pump, and has a bearing on each side of the casing. At these bearings the shaft is 8 ins. in diameter. The pump is driven by a cross-compound, condensing engine with an independent condenser. The high and low-pressure engines are horizontal, and are connected to two disc cranks set at right angles and keyed to opposite ends of the pump shaft. The high-pressure cylinder is 18 ins. and the low-pressure 32½ ins. in diameter; the stroke is 22 ins. The high-pressure cylinder has an adjustable cut-off, with a hand adjustment to regulate the point of cut-off. Expansion relief-valves are provided for the low-pressure cylinder. These engines develop about 500 H.-P., with a boiler pressure of 140 lbs. The jet pump is of the centrifugal type, with 18-in. suction and discharge. This pump supplies the water to stir up the sand at the suction head. The runner has four arms, with steel tips, running as close as practicable to the casing, to obtain the greatest available pressure. This pump and its engine rest on a common bed plate, and have a flange-coupled shaft for pump and engine. The engine is a compound condensing engine of the marine type. The high-pressure cylinder is 12 ins. and the low-pressure 22 ins. in diameter, with a common stroke of 14 ins. The crank shaft has the cranks set at right angles. The cylinders have plain slide valves. All parts are provided with automatic apparatus for lubrication.

The air pump is horizontal, with single steam cylinder 10 ins. in diameter, air cylinder 18 ins. in diameter, and 18-in. stroke. The con-

FIG. 1.

FIG. 2.

denser receives the steam from both main and jet pumps, and is so arranged that the steam can pass directly to the atmosphere. The overflow from the condenser passes through a chamber, forming a hot-well for the boiler feed pumps. There are two feed pumps and one fire pump, so arranged that any one of them can be connected with the feed-water pipe or the fire hose. These pumps are vertical, duplex, outside-packed plunger pumps, with 7½-in. steam cylinders, 4½-in. water plungers and 8-in. stroke. An ordinary sand filter, with alum as a coagulent, is provided to filter the water used in the galley and cabin. A separate pump forces the water through the filter to a tank on the roof, and from there it is piped to various parts of the boat.

The main hauling winches are located on opposite sides of the boat near the bow. They are driven independently and are provided with fast and slow feed controlled by clutches. The drums are 42 ins. in diameter and 42 ins. long, which gives space for 1 200 ft. of ¾-in. wire rope. Each drum is driven by a pair of 7 x 7-in. double cylinder engines with cam reverse, and the power is transmitted through a combination of worm and spur gearing. The movement is controlled by means of a brakewheel and strap. The speed ratio is 81 to 1 for fast and 493 to 1 for slow speeds. The hoisting engine for raising the suction pipe has a drum 24 ins. in diameter and 24 ins. long. It is fitted with brake, spur gear, pinion, clutch and shifting device. This engine also operates a shaft terminating in gipsy heads, which can be used in lieu of the capstans. The whole is driven by a pair of horizontal, double-cylinder, 8 x 10-in., plain, slide-valve engines, provided with steam reverse.

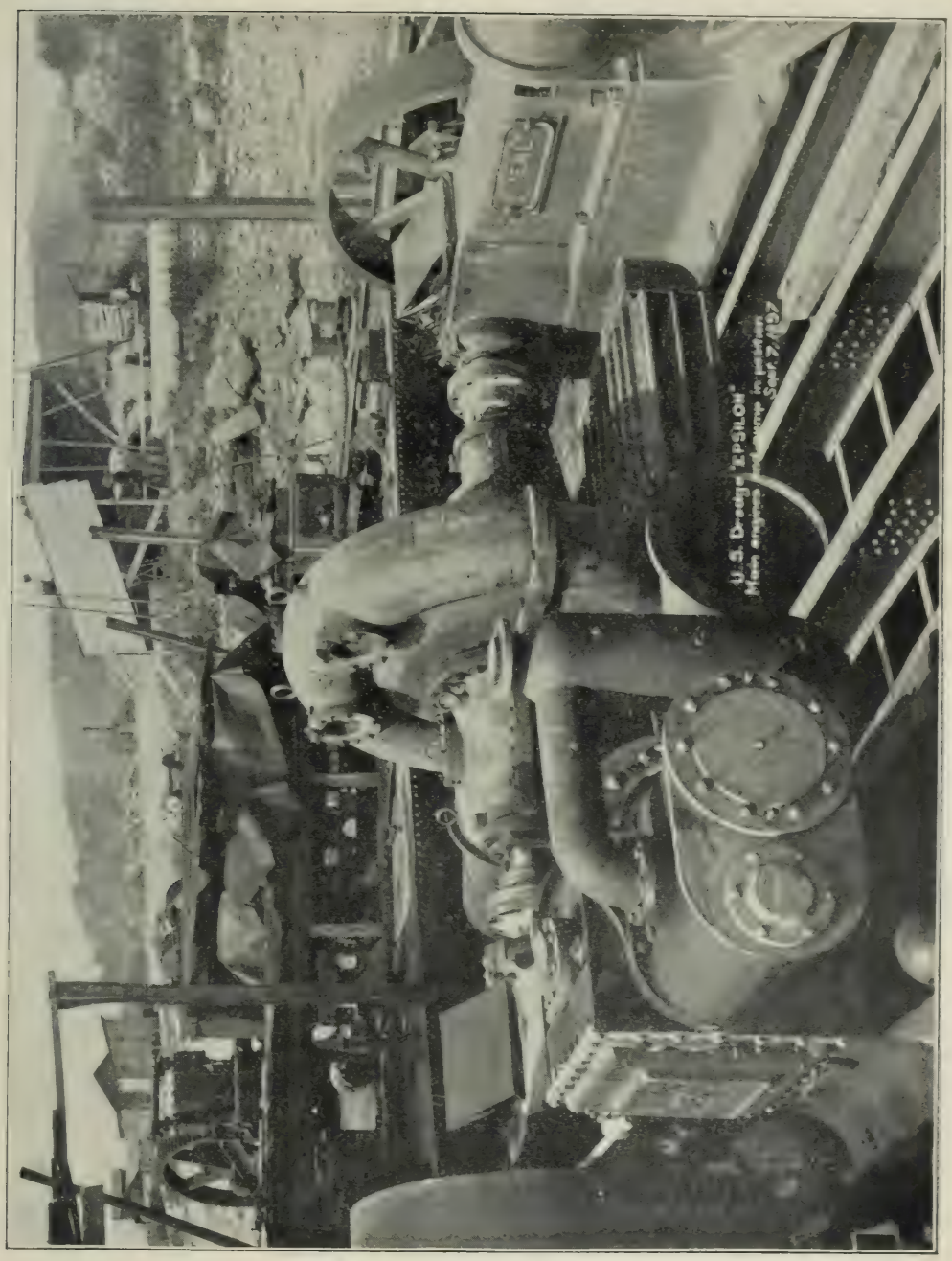
The anchor spud is hoisted by means of an upright, single-acting, steam cylinder, 18 ins. in diameter and 6-ft. stroke. The piston is attached to a cam clamp at its upper end, which grips and holds the spud against the slides. Steam enters at the bottom of the cylinder through a three-way valve, raising the piston and spud to the height of its movement. When the valve is turned to the exhaust port the piston and clamp descend, the spud being supported by a rack and pinion. The pawl locking the pinion is hinged between its center of movement and point, and the tripping is effected by making the pawl buckle on itself so that the point turns under. This releases its hold and allows the spud to drop.

Steam for the above-described plant is furnished by six boilers 48 ins. in diameter and 28 ft. long, with five 11-in. flues in each. These

boilers are arranged in two batteries so that they can be used separately if desired. They are designed to carry a pressure of 140 lbs. The fire-bed and ash-pan rest directly on the keelsons. Directly in front of the fire doors are coal bunkers having a capacity of 40 tons. The bottoms of the mud drums are below the water-line, hence a special cleaning device is provided by means of a tank set between the keelsons under the mud valves. The mud and water remaining in the drums are drawn off into the tank, and then syphoned out. The feed water is heated by passing through two pipes hung in brackets from the sides of the parting wall between the two battery furnaces.

The two intake pipes at the pump casing run forward to the suction head. They are hinged to the boat at the bow by two flanges connected by a pivot pin on the lower side and provided with a circular stuffing-box, making a radial joint that will allow the suction to be lowered to a depth of 15 ft. Outside of the joint the pipes and suction head are framed together and rigidly connected so as to form essentially one piece. At the suction head the pipes are 11 ft. apart from center to center. The suction head for each pipe is 8 ft. long and they are 3 ft. apart, forming one suction head 19 ft. long over all. The suction head proper has two inlets, one for the upper and one for the bottom intake, and between the two is a triangular pressure chamber running the whole length of the head. This connects directly with the jet pump. This chamber has nine $2\frac{1}{4}$-in. nozzles through which the water from the jet pump is forced with sufficient pressure to loosen the sand in front of the suction. The discharge pipe is 34 ins. in diameter. It runs from the bottom of the pump casing along the floors between the two central keelsons, under the partition walls between the batteries of boilers, and through the stern of the hull, with the center of the pipe 4 ft. above the bottom of the hull at its exit. Near the pump the entire pipe is below the water-line, and the priming is done by means of a steam syphon at the top of the pump casing. The floating discharge pipe is 1 000 ft. long, divided into 20 sections, each 50 ft. long. They are made of $\frac{1}{4}$-in. tank steel. The pipes are floated by means of 40 cylindrical pontoons, each 23 ft. 9 in. long and 30 ins. in diameter. These are made of $\frac{3}{16}$-in. tank steel. They are attached to the discharge pipe by means of truss frames constructed of bar and angle irons, so designed as to receive and clamp the discharge pipe and carry the pontoons on either side. The pipe line is

PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XL, No. 838.
OCKERSON ON DREDGES AND DREDGING.



attached to the hull section of the discharge pipe by means of a male and female bevel-flanged coupling, a davit being used to swing the floating pipe into position.

The hull has a cabin 97 ft. long and 29 ft. wide, which protects the machinery and provides quarters for the crew. There is also a repair shop, provided with a lathe, drill press, emery grinder and other appliances needed in making ordinary repairs. An electric plant furnishes light for one 4 000-candle-power search-light, four arc-lights of 1 200-candle-power each, and 75 incandescent lamps of 16-candle-power each.

All the winches, the spud hoist and the search-light are manipulated from the operating room. This dredge is towed from point to point, and while dredging is operated similarly to the *Alpha*.

Four 10-in. and four 12-in. hydraulic piles 33 ft. long are provided, similar to that shown in Fig. 7. A general elevation of this dredge is shown in Fig. 2, Plate XXV, and the arrangement of the machinery, etc., may be seen in Fig. 10.

This dredge was tested near St. Louis, Mo., in August, 1897, and, after the completion of the tests, she was used in actual dredging between Cairo and Memphis until early in December of the same year. The general efficiency test, called for by the contract, required that the dredge should be operated 60 working days of 12 hours each in water from 5 to 15 ft. deep, and with sand of such different degrees of coarseness as will be found on the low-water bars. After this had been done and the machinery found satisfactory, then 20 capacity tests were required to be made with the suction at different depths. This last requirement was considered filled when the total amount pumped per hour divided by 20 was equal to or exceeded the required capacity of 800 cu. yds. per hour.

The weights of the principal parts are as follows: Hull, 328 328 lbs.; cabin, 117 449 lbs.; main pump and engines, 74 722 lbs.; jet pump and engines, 21 717 lbs.; auxiliary engines, drums, levers, etc., 46 937 lbs.; air, feed and fire pumps, 15 695 lbs.; electric light plant, 7 700 lbs.; boilers and accessories, 325 911 lbs.; suction head, 49 534 lbs.; floating discharge pipe and pontoons, 244 916 lbs.; hull fittings, 31 580 lbs. Total, 1 019 623 lbs. The working draft of the dredge is about 46 ins.

Construction was begun in June, 1896, and the dredge was practically completed in March, 1897.

The results of the official tests are given in Table No. 4.

TABLE No. 4.—Capacity and Efficiency Tests of United States Dredge *Gamma*.

Number of test.	Duration of test.	Conditions.[1]		Forward movement of dredge per minute.	Main Engine and Pump.										Jet Engine and Pump (Nine jets, 2¼ ins. diameter.					
		Depth of water.	Depth of suction.		Steam pressure, pounds per square inch.	Vacuum in condenser, inches of mercury.	Revolutions per minute.	I. H.-P.			Head.				Revolutions per minute.	I. H.-P.			Head.	
								High-pressure cylinder.	Low-pressure cylinder.	Total.	Suction, feet of water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.		High-pressure cylinder.	Low-pressure cylinder.	Total.	Suction, feet of water.	Discharge, feet of water.
	Seconds.	Feet.	Feet.	Feet.																
1...	80.4	5.2	14.0	2.0	143	25.5	162	239.8	265.6	505.4	10.3	29.3	2.1	41.7	148	60.4	26.8	87.2	3.7	31.6
2...	90.8	7.0	14.0	3.9	135	25.0	176	212.7	266.6	479.3	8.0	35.0	1.8	44.8	146	56.6	26.0	82.6	4.0	31.6
3...	105.3	4.7	12.0	2.2	152	25.5	174	229.7	256.5	486.2	8.0	38.5	1.1	47.6	143	54.1	23.5	77.6	3.7	31.1
4...	81.2	4.0	11.0	1.8	162	25.0	163	215.6	221.3	436.9	7.5	30.9	2.0	40.4	144	55.3	24.5	79.8	3.7	30.6
5...	59.5	5.2	12.0	4.5	144	25.5	172	242.2	270.0	512.2	9.4	35.0	1.7	46.1	146	55.6	26.4	82.0	3.7	31.3
6...	40.6	4.3	12.0	5.2	139	26.0	165	228.6	249.0	477.6	9.6	38.7	1.5	49.8	144	53.0	24.2	77.2	3.8	33.4
7...	64.7	4.3	12.0	3.5	145	26.5	166	.....	.....	.....	9.2	41.5	1.0	51.7	142	49.4	22.4	71.8	3.8	31.3
8...	67.2	4.8	11.0	5.6	130	26.0	170	225.3	257.3	482.6	8.6	41.5	1.4	51.5	158	63.7	31.0	94.7	4.0	31.6
9...	69.7	5.8	15.0	2.3	140	24.0	166	240.0	240.6	480.6	8.0	33.9	1.8	43.7	160	66.1	32.7	98.8	5.2	34.0
10...	97.6	5.2	15.0	3.2	143	25.0	167	.....	.....	.....	14.3	45.4	0.4	60.1	165	74.1	38.3	112.4	4.2	36.8
11...	64.4	5.2	11.0	6.6	125	26.7	169	225.4	282.1	507.5	10.3	37.3	1.8	49.4	160	68.1	34.7	102.8	4.2	38.2
	74.67	5.06	12.6	3.71	141.6	25.52	168.2	228.81	256.56	485.37	9.38	37.00	1.51	47.89	150.5	59.67	28.23	87.90	4.00	32.86
SPECIAL TEST.																				
12...	63.8	6.3	11.0	2.2	142	24.0	170	239.0	265.2	504.2	8.6	30.4	2.6	41.6	.....	.....	.....	.....	.....	.....

TABLE No. 4—(*Continued*).

Number of test.	Measuring Barge (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.)										Efficiency.					
	Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Quantity of material pumped.	Quantity per second.	Velocity of discharge, per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, per I. H.-P.	Weight of a cubic foot of dry sand.	Percentage of voids.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu.yds.	Ft.-lbs.	Ft.-lbs.		Cu.yds.	Lbs.	
1..	0.487	2.783	2.296	5 874.8	73.07	11.59	0.332	840.8	14.3	1 394.0	190 438	277 970	68.5	2.76	98.5	37.2
2..	0.421	2.800	2.379	6 089.7	67.07	10.64	0.379	961.5	16.0	1 411.9	187 796	263 615	71.2	2.95	99.0	36.7
3..	0.767	2.967	2.200	5 629.1	53.46	8.48	0.352	892.0	15.8	1 129.5	159 043	267 410	59.5	2.32	105.5	32.8
4..	0.375	2.650	2.275	5 821.0	71.69	11.37	0.187	473.6	8.1	776.2	181 017	240 295	75.3	1.78	99.0	37.3 [2]
5..	0.417	1.967	1.550	3 966.0	66.66	10.57	0.347	878.8	22.2	1 967.4	192 064	281 710	68.2	3.84	103.5	35.7
6..	0.317	1.296	0.979	2 507.5	61.76	9.79	0.214	542.0	21.6	1 780.0	192 228	262 680	73.2	3.73	101.5	34.6
7..	0.367	1.617	1.250	3 198.4	49.43	7.84	0.270	683.8	21.4	1 410.9	159 721 [4]	.....	.....	.....	103.0	34.4 [3]
8..	0.290	1.850	1.560	3 991.6	59.40	9.42	0.436	1 101.0	27.6	2 184.5	191 194	265 430	72.0	4.53	105.0	34.0
9..	0.267	2.117	1.850	4 733.6	67.91	10.77	0.247	625.5	13.2	1 194.6	185 479	264 330	70.2	2.49	99.5	33.2
10..	0.345	1.583	1.238	3 167.7	32.46	5.15	0.263	666.0	21.0	909.6	121 928 [4]	.....	.....	.....	107.0	32.8 [5]
11..	0.379	2.067	1.688	4 319.1	67.07	10.64	0.495	1 253.2	29.0	2 595.2	207 079	279 125	74.2	5.11	106.0	36.3
	0.403	2.1543	1.7514	4 481.68	60.907	9.660	0.3202	810.75	19.11	1 523.09	187 370.9	266 951.7	70.26	3.28	102.5	35.0 [6]
SPECIAL TEST.																
12..	0.533	2.575	2.042	5 224.9	81.89	12.99	0.136	344 0	6.6	718.9	212 914	277 310	76.8	1.43	100.0	36.9 [7]

[1] Suction pipes: Two, 24 ins. diameter; length from mouth to pump = 48 ft. Discharge pipe: Diameter, 34 ins.; length from stern of dredge to valve in barge = 987.1 ft. Length from pump to valve in barge = 1 069 ft. Advance of dredge per revolution of winding engines = $\frac{1}{4}\frac{1}{3}$ ft. [2] Experiment with light load. [3] No indicator cards were taken. [4] Not used in computing average. [5] Pipes sunk at test. No indicator cards were taken. [6] Average. [7] Jets not in operation

Dredge *Delta*.

The dredge *Delta* was constructed under contract with the New York Dredging Co., which sublet the construction of different parts to various manufacturers and builders of machinery. The contract price for this dredge was $124 940. It differs from the *Gamma* chiefly in having a mechanical agitator instead of jets, in its hoisting and hauling winches, and in the form of pump. Its capacity is also somewhat greater. Its construction was begun in June, 1896, and it was practically completed by the end of June, 1897. After the tests were finished, this dredge was used in removing sand bars below Cairo throughout the low-water season of 1897.

The hull is of steel, 175 ft. long, 38 ft. wide, and 8¼ ft. deep. A fender 24 ft. long is carried around the suction, making the boat 199 ft. long over all. The bow and stern of the hull have short rakes, and the midship section is rectangular. The frames of the hull are 22 ins. apart between centers. Angles are used for the deck beams and Z-bars for the floors. There are two longitudinal bulkheads running the full length of the boat, and five cross-bulkheads. They are all made of $\frac{5}{16}$-in. plate, with double angles at top and bottom. All are watertight and have a syphon in each of the compartments. The thickness of the side plating of the hull is $\frac{3}{8}$ in. and of the other hull plating $\frac{5}{16}$ in. It is all laid fore and aft, is lapped and single-riveted, except on deck, where the seams are planed and butted and fastened with 6-in. butt-straps single-riveted. Cross-seams are similarly butted and riveted.

Special foundations of 12-in. I-beams are provided for sustaining the heavy machinery and boilers. The boat is provided with a cabin 156 ft. long, which protects the machinery and furnishes quarters for the crew. The operating room is at the forward end of the cabin, and is fitted with levers and brakes by means of which the boat is maneuvered.

The dredging pump is different from those on the dredges heretofore described, in the shape of the casing and the runner. The runner has five blades 22 ins. wide, and is 7 ft. in diameter. The edges run close to the casing, but the runner is not concentric with the casing, hence the outer ends of the arms are nearer one side of the casing than the other, the widest space being at the bottom, and the space being nearly cut off by a projection in the casing at the upper

HYDRAULIC DREDGE GAMMA

DIMENSIONS.

LENGTH OF HULL,	105′6″	DIAM. OF CENTRIFUGAL SAND PUMP,	69″
LENGTH OVER ALL,	138′0″	DIAM. OF SUCTION AND DISCHARGE PIPE,	34″
BREADTH OF HULL,	38′0″	LENGTH OF DISCHARGE PIPE,	1000′ 0″
DEPTH OF HULL,	8′2″	DIAM. OF CENTRIFUGAL JET PUMP,	69″
WORKING DRAFT,	3′10″	NUMBER OF BOILERS = 6	48″ X 28′

AVERAGE CAPACITY 1523 CU. YDS. OF ORDINARY RIVER SAND PER HOUR THROUGH 987 FT. OF FLOATING PIPE.

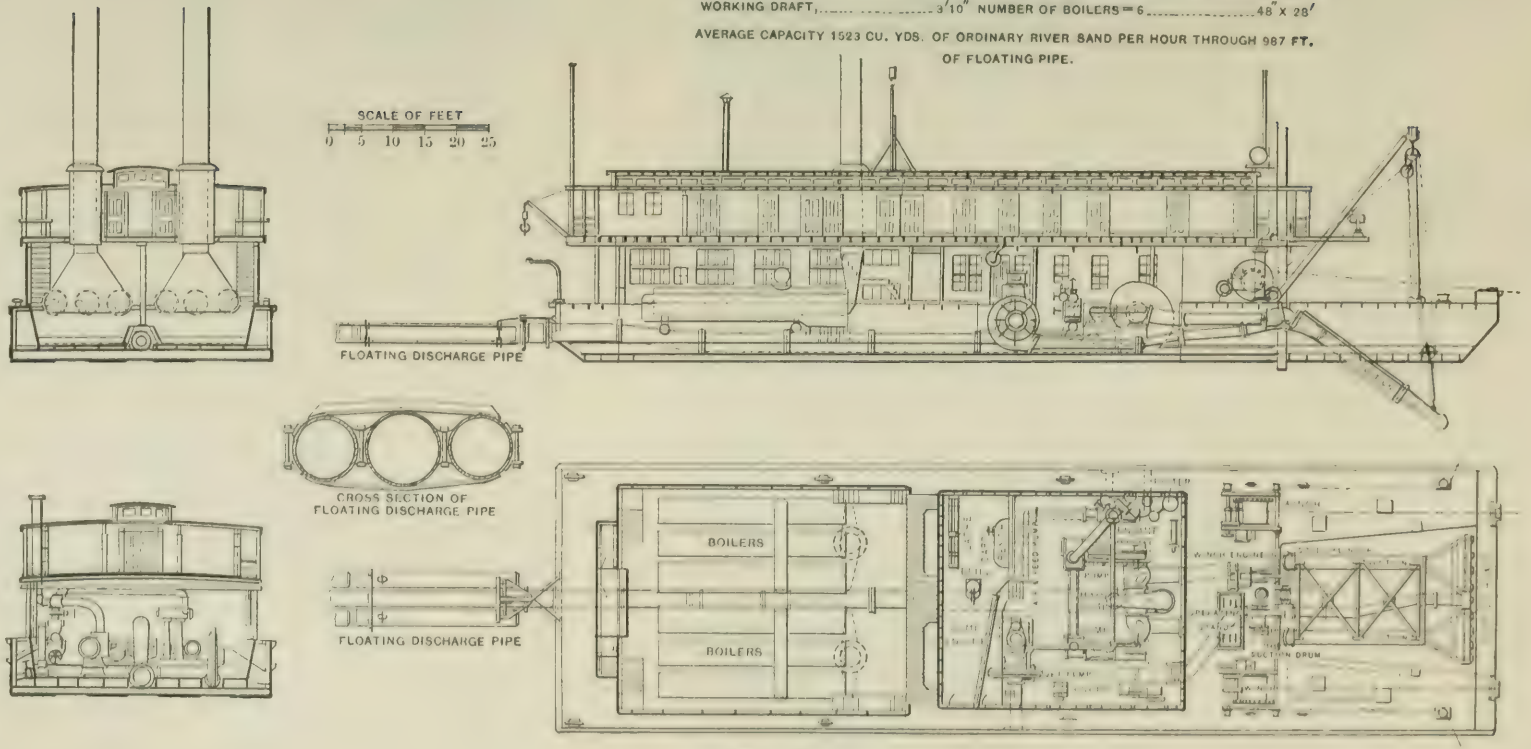


FIG. 10.

side of the discharge opening. The shape of the pump runner and the upper half of the casing is shown in Fig. 1, Plate XXVI. The axis of the pump is parallel to the axis of the boat and lies over the center line of the same. The thickness of the sides of the casing is 2 ins., and that of the outer circumference is 3 ins. It is lined on the inner side with steel plates fastened with countersunk bolts. This lining can be renewed when worn out. The shaft of the runner is of forged steel $7\frac{1}{2}$ ins. in diameter and $7\frac{1}{2}$ ft. long. It is fitted to the runner by a taper joint secured by a cap nut and two keys. The shaft has one long bearing through the aft side of the casing. It is provided with water bushing under pressure to keep the sand out of the bearing. The sand pump is driven by a vertical, inverted, two-crank, compound-condensing engine, with 22 and 48-in. cylinders and 24-in. stroke. It is fitted with a piston and slide valve and has an adjustable cut-off for the piston valve. The cylinders are supported by two cast-iron back frames of box section, and four taper-steel front columns $2\frac{3}{4}$ and $3\frac{3}{4}$ ins. in diameter. The bed plate is of cast iron, box-girder pattern, 8 ft. 9 ins. long by 8 ft. $9\frac{1}{2}$ ins. wide over flanges, and $20\frac{1}{2}$ ins. deep from center of shaft. It has four babbitted journals 13 ins. long and $8\frac{1}{2}$ ins. in diameter. The steam chests are reached by means of a stairway and gallery. The usual accessories in the way of relief valves, drains, lubricators, etc., are provided. This engine was designed to develop 800 H.-P. at 140 revolutions per minute, with a boiler pressure of 160 lbs. and a vacuum of 25 ins.

The air pump and jet condenser has a steam cylinder 12 ins. in diameter, a water cylinder 18 ins., and 24-in. stroke. The condenser is mounted vertically over the water cylinder. The air cylinder is copper-lined 18 by 24 ins., and the plunger is packed with soft packing. The valve seats are brass and the valves are of hard rubber. A brass spray-cylinder throws the water out in jets.

The engine which drives the cutters is horizontal, two-cylinder and non-reversible, all attached to a sliding steel frame, which moves back and forth in guides as the cutter is raised or lowered. This is necessary because the shaft which drives the sprocket chain is not in the axis of motion on which the suction and cutter revolve. The whole engine, with its frame, follows the motion of the shaft, so that the gear and pinion are always engaged. To admit of this motion, the steam pipes are provided with slip joints. The cylinders of this engine are

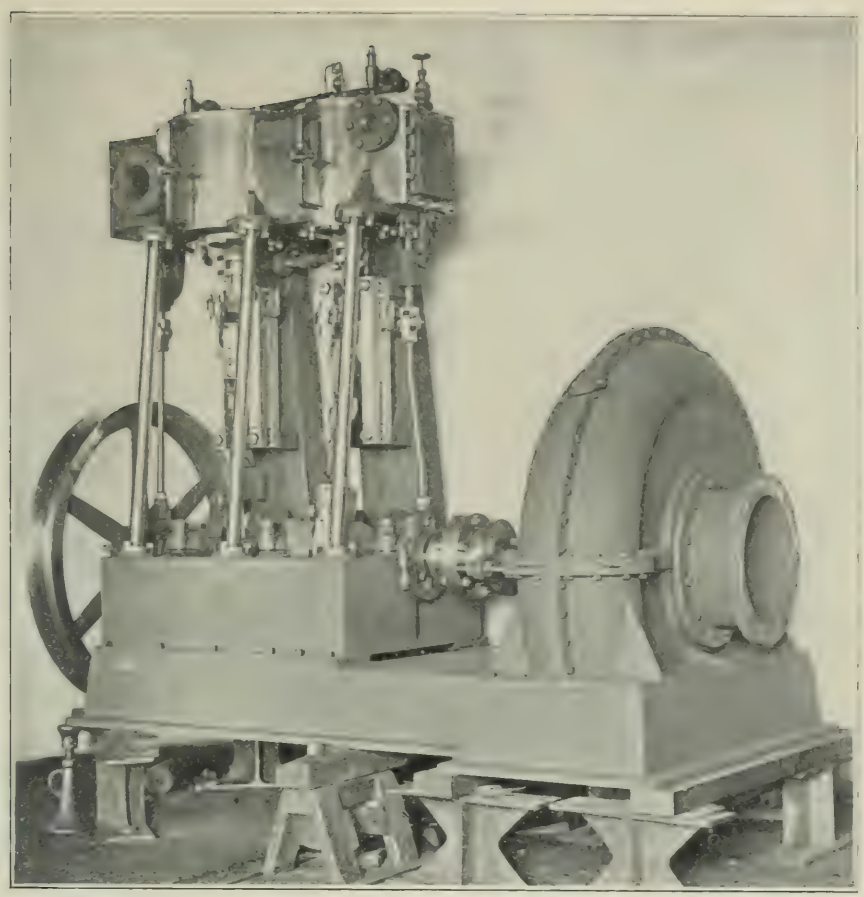


FIG. 1.

FIG. 2.

12¼ ins. in diameter and have a 15-in. stroke, with the locomotive type of slide valve. It operates the cutters through spur gears and a sprocket chain.

The fire pump is duplex, with two steam cylinders 8 ins. in diameter and two 5-in. plungers with a stroke of 10 ins. The suction is 5 ins. in diameter, and the delivery 4 ins. This pump furnishes water to all the journals where water pressure is used to keep out sand. The feed pump is also duplex, with two steam cylinders 7 ins. in diameter, two plungers 4½ ins. in diameter, with a stroke of 8 inches.

A pressure filter 7 ft. in diameter, with a capacity of about 65 galls. per minute is provided. A special 3 by 4-in. engine is used to revolve the cleaners.

A 7 by 9-in. vertical engine is provided to run the machines in the repair shop and the electric generator.

There are two winding drums located forward of the sand pump, one on the starboard and the other on the port side of the hull. These are 42 ins. in diameter and 42 ins. long, mounted on a 6-in. shaft with two bearings in the main pillow-blocks carrying a 48-in. gear wheel at one end, and an 84-in. gear wheel at the other. These drums are provided with clutches and brakes, and are driven by two independent double-cylinder, horizontal engines with 10-in. by 12-in. cylinders.

The ladder hoist for raising and lowering the suction, and the spud hoist for raising the spud, each have drums 24 ins. in diameter and 24 ins. long. The former is on the starboard, and the latter on the port side of the boat. The cables from these drums lead to the roof and thence out through sheaves to the ladder and spud. These drums are operated by the same engines that operate the winding drums.

Steam is supplied by four Heine safety boilers rated at 250-H.-P. each. These boilers have two shells 36 ins. in diameter and 19 ft. 4 ins. long; two water legs of flanged and riveted sheets held by hollow stay bolts; 140 lap-welded tubes 3½ ins. in diameter and 16 ft. long, and one submerged movable mud drum in each shell. These boilers are erected in two batteries on the main deck near the aft end of the boat. For each battery there is one stack, 68 ins. in diameter and 64 ft. high above the grate. The feed water is heated by passing through a heater with a shell 3 ft. 1½ ins. in diameter, fitted with 114 corrugated copper tubes 1½ ins. in diameter and 78 ins. long. This heater is situated in the center of the boat between the boilers and the

TABLE No. 5.—Capacity and Efficiency Tests of United States Dredge *Delta*.

Number of test.	Duration of test.	Conditions.[1]		Speed of cutters, revolutions per minute.	Forward movement of dredge per minute.	Main Engine and Pump.									
		Depth of water.	Depth of suction.			Steam pressure, pounds per square inch.	Vacuum in condenser, inches of mercury.	Revolutions per minute.	I. H.-P.			Head.			
									High-pressure cylinder.	Low-pressure cylinder.	Total.	Suction, feet of water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.
	Seconds.	Feet.	Feet.		Feet.										
1.....	77.0	6.0	11.2	7.7	3.9	159	24.0	150	498.5	536.6	1 035.1	17.0	46.4	4.1	67.5
2.....	62.5	5.8	11.2	9.0	4.4	156	23.3	151	.....	.....	.....	18.7	45.3	4.2	68.2[2]
3.....	71.5	7.0	12.1	7.6	6.0	156	23.0	149	.....	.....	.....	18.1	45.3	3.8	67.2[3]
4.....	79.0	6.7	12.1	9.3	3.8	156	23.0	151	496.7	534.2	1 030.9	19.2	54.5	2.3	76.0[4]
5.....	75.5	8.3	14.2	7.7	2.7	156	23.5	150	491.7	505.2	996.9	18.1	47.6	3.5	69.2
6.....	61.0	5.3	12.5	9.7	4.2	160	23.0	149	532.4	566.2	1 098.6	19.8	45.3	3.4	68.5[5]
7.....	59.25	.....	.....	.....	5.1	161	23.0	152	492.0	490.1	982.1	19.2	47.6	3.0	69.8[6]
8.....	61.0	7.0	12.7	9.4	4.4	160	24.2	149	461.9	456.3	918.2	17.0	47.6	3.7	68.3[7]
9.....	60.0	7.8	13.3	9.2	5.4	161	23.5	150	536.6	553.3	1 089.9	23.2	38.4	3.0	64.6
10.....	60.5	7.0	15.0	7.0	3.7	156	23.3	149	.....	.....	.....	19.2	43.0	3.5	65.7[8]
11.....	58.0	6.3	15.0	8.2	2.5	161	23.0	152	543.1	553.7	1 096.8	18.1	41.8	3.7	63.6
12.....	60.5	10.0	13.5	7.9	4.3	156	23.8	151	515.5	530.3	1 045.8	17.5	43.0	3.3	63.8
13.....	61.0	9.5	14.8	7.4	2.6	156	23.0	154	517.4	554.9	1 072.3	17.5	45.3	3.4	66.2[10]
14.....	72.0	10.0	14.5	7.9	2.8	156	23.0	154	522.3	546.9	1 069.2	18.1	46.4	3.5	68.0[11]
	65.63	7.44	13.24	8.30	3.99	157.9	23.33	150.8	509.8	529.8	1 039.6	18.62	45.54	3.46	67.61[12]
				7.96	3.95	157.6	23.31	151.1	514.6	533.9	1 048.5	18.61	44.55	3.55	66.71[13]
SPECIAL TESTS.															
15.....	53.0	10.5	13.5	9.6	5.2	156	22.8	146	511.0	523.3	1 034.3	21.5	33.8	5.3	60.6[14]
16.....	60.0	8.0	13.3	.....	3.8	161	23.2	149	490.4	501.9	992.3	18.1	38.4	3.9	60.4[15]

Number of test.	Measuring Barge. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.) Mean depth of water in barge before test.	Mean depth of water and sand in barge at end of test.	Mean depth of material pumped.	Quantity of material pumped.	Quantity per second.	Velocity of discharge, per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Efficiency. Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, per I. H.-P.	Weight per cubic foot of dry sand.	Percentage of voids in wet sand.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu. yds.	Ft.-lbs.	Ft.-lbs.		Cu. yds.	Lbs.	
1....	0.396	3.467	3.071	7 857.8	102.05	16.19	0.398	1 007.9	12.8	1 745.3	430 523	569 305	75.6	1.69	98.5	37.2
2....	0.417	2.958	2.541	6 501.7	104.03	16.50	0.300	759.8	11.7	1 620.9	443 428[9]	.....	.....	.....	76.5	37.2[2]
3....	0.338	3.083	2.745	7 026.2	98.27	15.59	0.471	1 192.8	17.0	2 224.3	412 734[9]	.....	.....	.....	94.5	37.2[3]
4....	0.496	2.850	2.354	6 023.2	76.24	12.09	0.427	1 080.5	17.8	1 823.7	362 140	566 995	63.9	1.77	98.5	34.0[4]
5....	0.383	3.167	2.784	7 123.4	94.35	14.96	0.208	526.8	7.4	928.0	408 064	548 295	74.4	0.93	104.5	35.2
6....	0.571	2.783	2.212	5 659.8	92.78	14.71	0.439	1 111.8	19.6	2 432.4	397 214	604 230	65.7	2.21	108.5	36.3[5]
7....	0.362	2.408	2.046	5 235.1	88.36	14.01	0.518	1 311.8	25.1	2 949.2	385 420	540 155	71.4	3.00	100.5	40.0[6]
8....	0.583	2.900	2.317	5 928.5	97.19	15.41	0.180	455.8	7.7	995.0	414 880	505 010	82.1	1.08	115.5	36.2[7]
9....	0.575	2.633	2.058	5 265.8	87.76	13.92	0.375	950.1	18.0	2 111.3	354 331	599 445	59.1	1.94	96.5	37.1
10....	0.429	2.667	2.238	5 726.4	94.65	15.01	0.576	1 457.7	25.5	3 212.4	388 656[9]	.....	.....	.....	110.5	33.4[8]
11....	0.450	2.650	2.200	5 629.1	97.05	15.39	0.317	802.8	14.3	1 853.2	385 773	603 240	63.9	1.69	100.5	37.5
12....	0.512	2.700	2.188	5 598.4	92.54	14.68	0.263	667.5	11.9	1 471.1	369 003	575 190	64.2	1.41	102.5	38.9
13....	0.342	2.567	2.225	5 693.1	93.33	14.80	0.224	567.3	10.0	1 240.0	386 152	589 765	65.5	1.16	96.5	35.5[10]
14....	0.421	3.100	2.679	6 856.3	95.23	15.10	0.215	544.7	7.9	1 000.9	404 727	588 060	69.0	0.94	112.5	35.4[11]
	0.4482	2.8524	2.4041	6 151.77	93.845	14.883	0.351	888.38	14.76	1 829.12	390 747.9	571 790.0	68.62	1.620	101.14	36.51[12]
	0.4205	2.8545	2.4341	6 228.48	95.238	15.105	0.3514	889.93	14.69	1 850.60	390 499.1	576 681.9	67.88	1.595	99.41	36.8[13]
SPECIAL TESTS.																
15....	0.308	2.716	2.408	6 166.5	116.35	18.45	0.242	613.3	9.9	1 542.9	440 675	568 865	77.5	1.49	92.5	33.4[14]
16....	0.467	2.817	2.350	6 012.9	100.22	15.90	0.227	574.0	9.5	1 277.5	378 330	545 765	69.3	1.29	110.5	34.9[15]

[1] Distance from pump to valve in barge = 1 104.5 ft. Distance from end of suction to pump = 84.6 ft. Forward movement of dredge per revolution of winding engines = $\frac{1}{23}$ ft. Ratio of speed of cutters to speed of cutter engine = $\frac{1}{23.9}$. [2] No indicator cards were taken. [3] No indicator cards were taken. [4] Test barge valve not completely open. [5] Test barge valve did not close and open completely. [6] Cutter machinery broke at test. [7] Test barge valve not completely open. [8] No indicator cards were taken. [9] Not used in computing averages. [10] In sand with a small percentage of gravel. [11] In sand with a small percentage of gravel. [12] Average. [13] Average, excluding tests Nos. 4, 6 and 8. [14] Discharge pipe 582 ft. long, using cutters. [15] Cutters removed from suction.

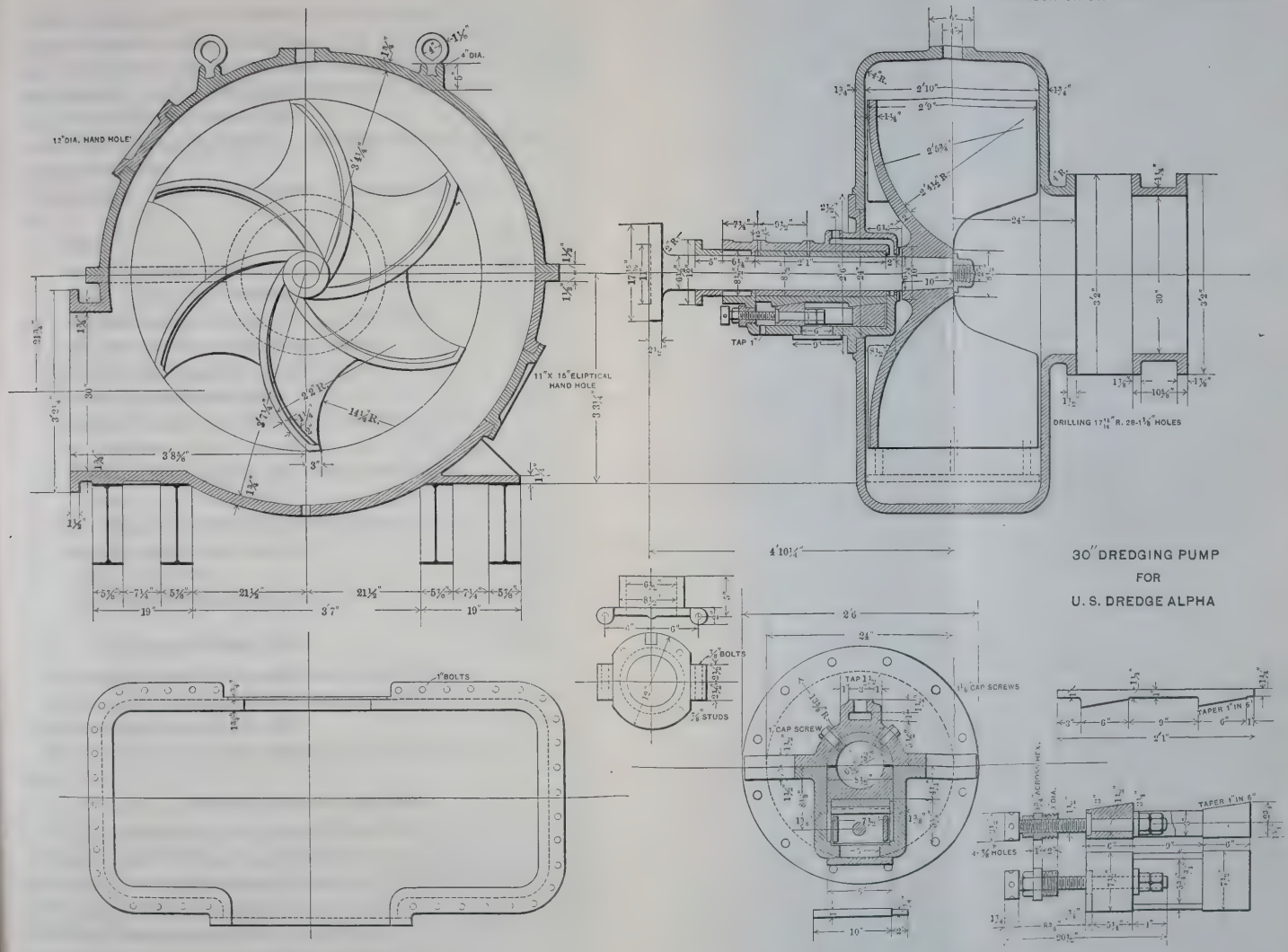
main engine. The main steam pipe is 6 ins. in diameter, the feed pipe 4 ins., and the drip, blow-off and safety pipes are 2 ins. in diameter.

The intake of the sand pump is from one pipe 34 ins. in diameter, entering in the axis of the pump at the forward side of the casing. This single pipe runs 25 ft. to the forward bulkhead and there branches into two pipes, each 24½ ins. in diameter, which separate and pass through the bow of the boat below the water-line 9 ft. apart. These two pipes turn to the right and left square along the outside of the bow and then turn forward again and each branch separates into two pipes 17½ ins. in diameter, which connect with the suction head. The whole is framed together so that the four pipes, suction head and cutters are raised and lowered together as one piece. Instead of a radial slip joint for the suction pipes, as used on the dredges already described, there is a vertical flanged joint in the horizontal part of each pipe next to the bow, and the revolving pins that sustain the weight of the aft end of the suction are placed in the prolonged axes of these pipes.

The cutter for loosening up the material is placed at the outer end of the suction head. It has twenty-two cast-steel wheel cutters, each having four blades mounted on a steel shaft 6⅞ ins. square. This shaft is driven by the cutter engine by means of two steel sprocket chains, at a rate of about eight revolutions per minute (see Fig. 2, Plate XXVI). The discharge outlet is on the starboard side of the casing near the bottom, and leaves the pump at right angles to the intake. After leaving the pump the pipe rises so that the center is slightly above the axis of the pump, and runs aft parallel to the side of the hull until it passes the main engine. Here it bends over to the center line of the hull, and the center of the pipe drops about 11 ins. below the water-line and runs straight aft under the boilers, through the stern, where the floating pipe is coupled on.

The floating discharge pipe is 1 000 ft. long, with the usual rubber couplings at intervals of 50 ft. There are pontoon floats on each side of this pipe, U-shaped in section, with the flat side closed, and they sustain the pipes in yokes which are firmly attached to the floats. There is a baffle plate at the end of the pipe line.

The dredge is provided with sixteen hydraulic piles, six of which are 10 ins. in diameter and 38 ft. long and ten are 6 ins. in diameter and 25 ft. long.


12" DIA. HAND HOLE
4" DIA.
11" X 15" ELIPTICAL HAND HOLE
14¼" R.
2'2" R.
3'8⅝"
3'7"
19"
21½"
TAP 1"
DRILLING 17/16" R. 28-1⅛" HOLES
4'10¼"
30" DREDGING PUMP
FOR
U. S. DREDGE ALPHA
1" BOLTS
⅞" BOLTS
⅞" STUDS
1" CAP SCREW
1⅛" CAP SCREWS
TAPER 1" IN 6"
4-⅞" HOLES

The various parts of the winding machinery can be handled and moved by means of a 5-ton traveling crane. A traveler is also provided, with which to move the pump and main engine. The repair shop contains a screw-cutting engine-lathe, a drill press, a blacksmith's forge and a full set of tools.

The boat is equipped with one search light, two arc lights and 100 incandescent lamps.

The weights of the various parts of the dredge are about as follows:

Hull	489 500 lbs.
Main pump and engine	105 568 "
Cutter engines and machinery	45 000 "
Winding engines and machinery, with extras	47 000 "
Air, feed and fire pumps, with extras	11 900 "
Electric plant engines, dynamos, etc	4 275 "
Heater, separator and filter	21 500 "
Derrick, ladder, travelers, tackle sheaves, bolsters, tools, etc	44 000 "
Capstans, spud, piping and fittings, deck fittings, etc	42 000 "
Discharge and suction pipes pertaining to hull	51 300 "
Boilers, stacks, etc., complete	320 000 "
Cabin, complete	180 000 "
Spare parts for main pump, etc	13 565 "
Total approximate weight	1 375 608 "

The weight of the floating pipe line is 221 477 lbs.

This dredge was launched on February 20th, 1897, and the tests were completed August 11th, 1897.

The results of the efficiency and capacity tests are given in Table No. 5.

Dredges *Epsilon* and *Zeta*.

The construction of the dredges *Epsilon* and *Zeta* was begun early in January, 1897. They were built under contract with the Springfield Boiler and Manufacturing Company, of Springfield, Ill. The work done by this company at its own shops was confined chiefly to the hulls, floating pipe lines, boilers, and other plate work. The pumps, engines, etc., were sublet to other manufacturers.

The stipulated capacity of these dredges is 1 000 cu. yds. of ordinary river sand per hour, dredged from a maximum depth of 15 ft., and delivered through 1 000 ft. of floating pipe.

The dredges are alike throughout, except that the *Epsilon* has water jet agitators and the *Zeta* has mechanical agitators. The contract price of the *Epsilon* is $102 000, and of the *Zeta* $106 000.

The hulls are of steel, 157 ft. long, 40 ft. wide and 7½ ft. deep. There is a well at the bow 35 ft. long, 18 ft. wide at the hull, and 22½ ft. wide at the forward end. There are two open spaces in the hull, one for the engine and pumps, and one for the boiler and coal bunkers. All other parts of the hull are decked over.

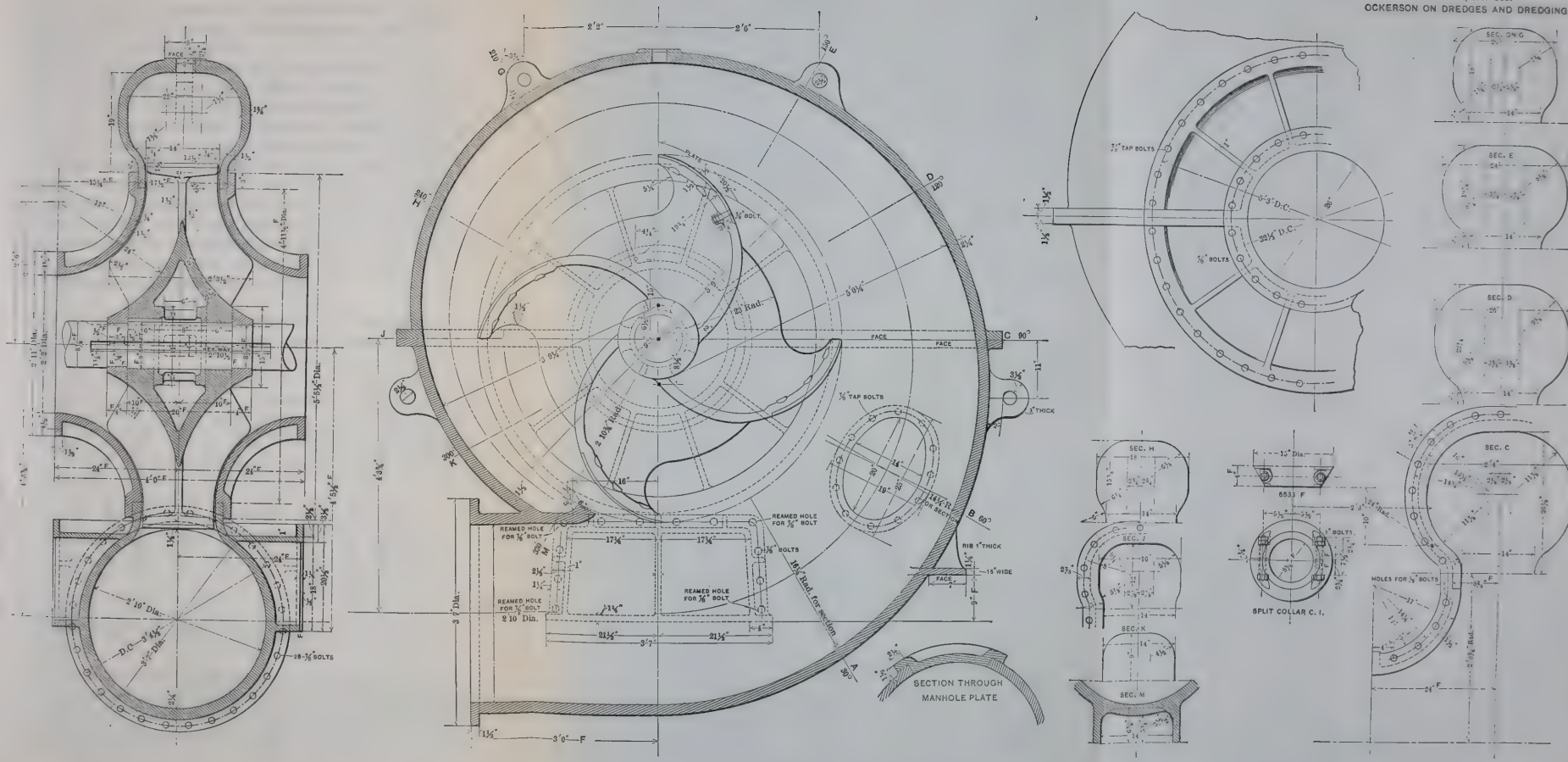
The floor beams are 12-in. steel channels, 25 lbs. per foot. The frames are 3 x 4-in. angle irons, 11 lbs. per foot, and are spaced 24 ins. between centers throughout the engine compartment, and 30 ins. for the other parts of the hull. There are ten 15-in. I-beam keelsons, the two outer ones weighing 55 lbs. per foot and the others 45 lbs. per foot. These keelsons are riveted to the floor beams.

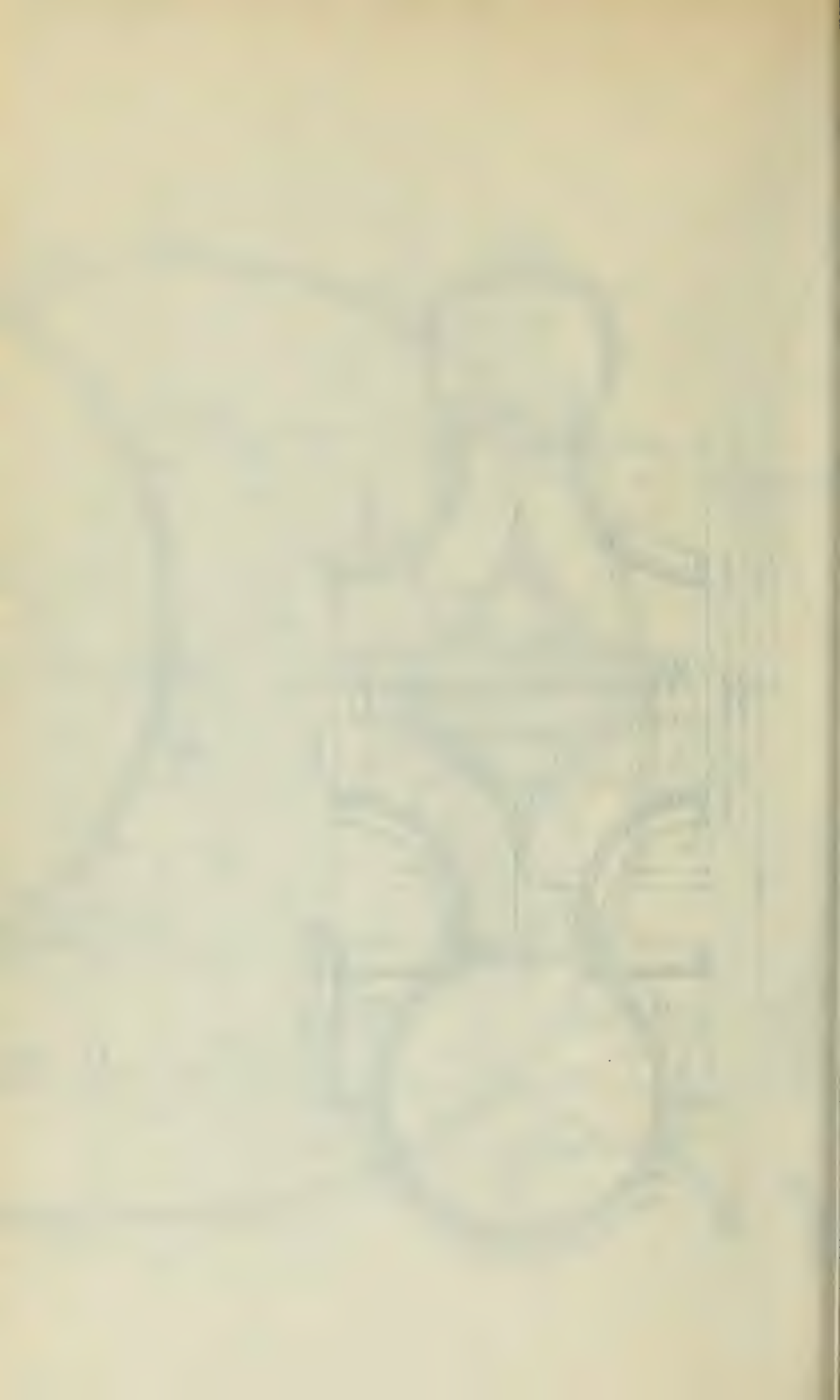
The greater part of the hull plating is ⅜ in. thick. The center strake of the bottom plating, the upper strake of the side plating and the bow end plates are ⅜ in. thick. The deck plating is ¼ in. thick. The butts of all outside plates are planed so as to fit closely. The longitudinal seams are lapped 2½ ins. and single riveted. The plates are butted on the transverse seams and riveted to butt straps ⅛ in. thicker than the plates.

The hull is divided into eleven water-tight compartments by two longitudinal bulkheads and five cross bulkheads.

The frames in the engine and boiler pits are covered with $\frac{3}{16}$-in. plates. Special beams and angles are provided for supporting the machinery.

The main sand pump is located on the axis of the hull, in the forward part of the engine pit. It is similar in form to that on the *Gamma*. That is to say, it has a divided suction, consisting of two 24-in. pipes, admitting the water on both sides of the casing, and the shaft of the runner extends through the casing with a bearing at each side. The main bearings of the pump runner are protected from sand by means of a ring of water under pressure supplied by the fire pump. The discharge is from the bottom of the casing and is 32 ins. in diameter. The pump runner is 5 ft. 9 ins. in diameter and has seven


SECTION THROUGH MANHOLE PLATE
SPLIT COLLAR C. I.
SEC. C
SEC. D
SEC. E
SEC. G
SEC. H
SEC. J
SEC. K
SEC. M
FACE
REAMED HOLE FOR ⅜" BOLT
⅜" TAP BOLTS
RIB 1" THICK
15" WIDE
HOLES FOR ⅜" BOLTS
1" BOLTS
KEY WAY
28-⅜" BOLTS



blades 11⅝ ins. wide at the outer ends. It is to be run at a speed of about 160 revolutions per minute. Pressure-gauges are provided to show the suction and delivery heads.

The main engines which operate the pump are connected to the ends of the shaft passing through the pump runner by means of flanged couplings. There is a tandem-compound engine at each end of the shaft, each having cylinders 16 and 26 ins. in diameter and an 18-in. stroke. The cranks are set at right angles with one another. These engines are balanced in a horizontal direction, and automatic governors are provided to regulate the speed. They are designed to develop 650 H.-P. at about 180 revolutions per minute, with a boiler pressure of 150 lbs. Plate XXVII show the engine in place. The engine sub-bases connect with and are bolted to the base of the pump casing, thus forming a common base for the whole. Table No. 6 shows the performance of these engines on the *Zeta*.

TABLE No. 6.—Showing Results of Tests of U. S. Dredge *Zeta*, at New Madrid, Mo., January, 1898. Size of Engine, 16 Ins. x 26 Ins. x 18 Ins.

Series.	Revolutions per minute.	Steam.	Suction head, water.	Delivery head, water.	Horse-Power.				Total horse-power.	Revolutions of cutter engines.
					Starboard engine.		Port engine.			
					High P.	Low P.	High P.	Low P.		
		Lbs.	Feet.	Feet.						
1.....	178	140	11.9	28.5	223	123	174	138	658	...
2.....	187	152	11.9	26.5	238	119	207	178	742	...
3.....	182	152	11.5	25.5	225	97	195	165	682	...
4.....	180	152	13.5	28.5	241	110	204	165	720	147
5.....	179	151	12.4	28.5	233	110	197	158	698	120
6.....	180	151	12.9	28.5	231	115	191	154	691	135
7.....	179	146	14.0	27.5	232	109	195	158	694	105
8.....	180	156	12.9	28.5	222	101	184	149	656	156
9.....	200	149	15.8	30.7	260	137	272	247	916	...

Notes.—

After taking Card No. 1, the governor was changed to a more sensitive position.

Shaft broke outside of main journal on starboard engine two minutes after cards were taken.

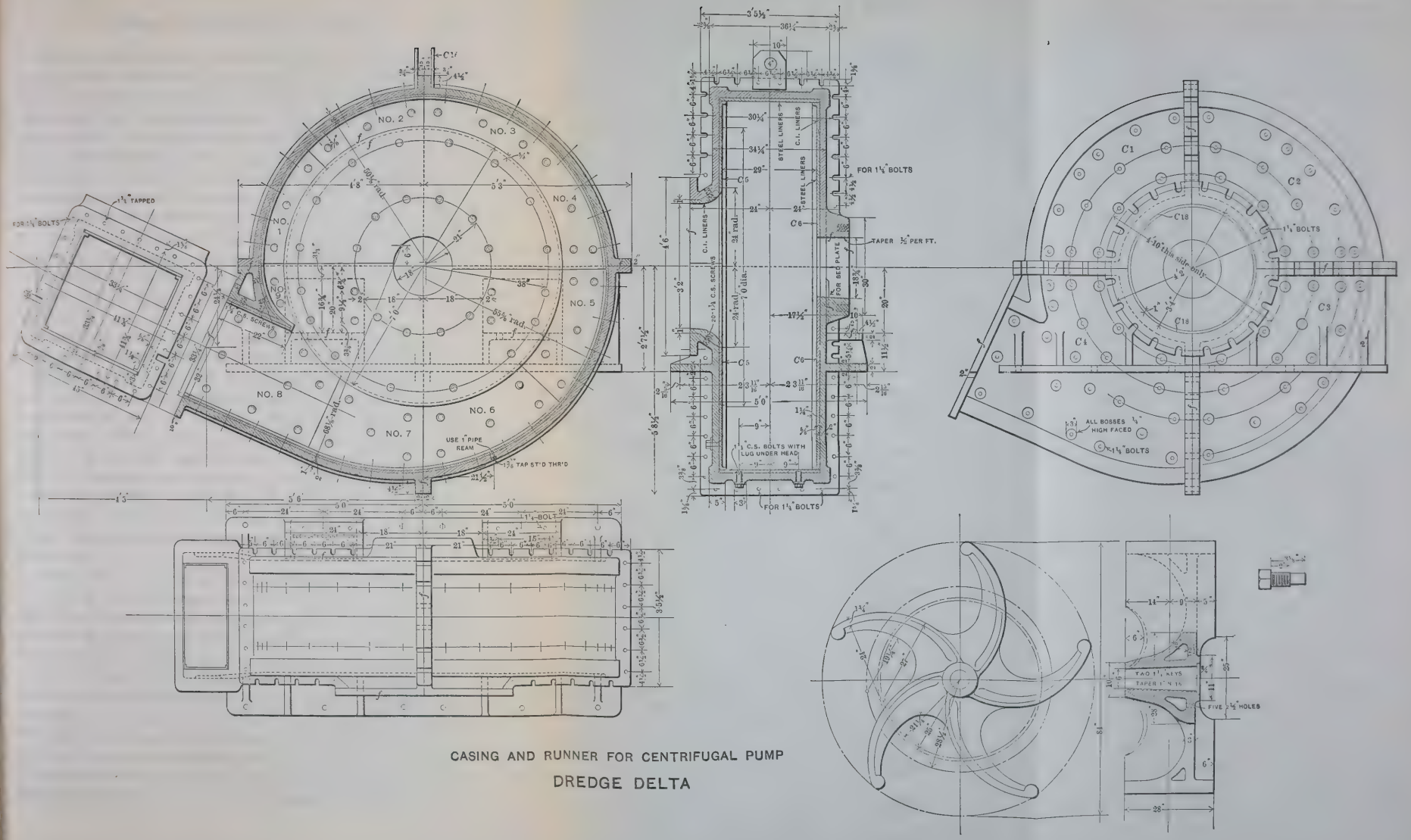
The *Epsilon* has a 15-in. centrifugal jet pump operated by a vertical cross-compound engine with cylinders 12 and 22 ins. in diameter and with 12-in. stroke. The engine is on the same base as the pump, and is directly connected with a flanged coupling. This pump is on the

starboard side at the rear end of the engine pit. The intake, 18 ins. in diameter, is on the side, and the discharge pipe, 15 ins. in diameter, leaves the casing at the bottom and leads to the pressure chamber at the suction head (Fig. 1, Plate XXVIII). Two boiler feed pumps are located in the engine pit opposite the jet pump, near the port side of the boat. The cylinders are 8 and 4½ ins. in diameter, and the stroke is 10 ins. There is also a fire pump, with cylinders 10 and 5 ins. in diameter, with a 10-in. stroke. These pumps are arranged with suitable cut-off valves so that they can be used together or singly, as may be desired.

The winding engines and the hoisting and hauling drums are located near the bow of the boat. The hauling engines for controlling the movements of the boat are located one on each side of the bow. The drums are 42 ins. long and 42 ins. in diameter, and hold about 1 200 ft. of ¾-in. wire rope in two laps. Each of these drums is operated by an 8 by 8-in. double-cylinder engine provided with a link reverse and suitable clutches and brakes giving fast or slow motion, as may be required. The suction hoist is situated a little to the port side of the center line of the hull. The drum is 14 ins. long, and 42 ins. in diameter. It is operated by a double engine with 8 by 8-in. cylinders. The spud hoist is located just behind the anchor spud and on the starboard side of the center line of the hull. It is also operated by an 8 by 8-in. double engine, and is used for raising or lowering the anchor spud. The drum is 21 ins. long and 42 ins. in diameter. The operator stands on an elevated platform behind the winches and controls their movements by means of vertical levers.

The repair shop is located between the operating room and the engine pit. It is provided with lathe, drill press, screw and bolt cutter, shaper, forge and an engine to run the several machines. A full set of machinist's and blacksmith's tools is also provided. In the same room is located the engine and direct-connected dynamo, furnishing light for all parts of the boat and also a powerful search light at the bow and two arc lights at the stern.

On the *Zeta* the jet pump is omitted and a mechanical agitator for stirring up the material is used. This agitator consists of a vertical scraper or harrow attached to the front end of the suction head which is given an up-and-down motion by means of a bell-crank. It is divi-



CASING AND RUNNER FOR CENTRIFUGAL PUMP

DREDGE DELTA

ded into two parts, which are suspended in front of the port and starboard suctions, respectively. A connecting rod or pitman extends from each agitator back to the rear end of the well, where it connects with a second oscillating crank operated by an engine with 10-in. by 14-in. cylinders. These engines are connected through gearing to a common shaft extending entirely across the boat. In order to lessen the danger of breakage from snags or other solid obstructions, a shearing pin is provided at the angle of the forward crank which is so arranged as to shear off before sufficient strain can come on the agitator to break it. Mechanically considered, its movements are quite satisfactory. In the tests it was demonstrated that the capacity of the pump with this agitator was not much over half the capacity of the pump with jet agitators. It will therefore be taken out, and jets will be put in.

Steam for the various machines described above is derived from six boilers located near the aft end of the boat. These boilers are 48 ins. in diameter and 28 ft. long, with three 11-in. and two 13-in. flues, and are designed for a working pressure of 140 lbs. They are arranged in two batteries so as to be operated separately or together. Each battery has a smokestack 44 ins. in diameter and 70 ft. high. The boilers are set below the main deck and rest on channel bars placed across the keelsons. The feed water is pumped through a heater which receives the exhaust steam from the main engines. A donkey boiler, 48 ins. in diameter and 9 ft. high, is also provided to furnish steam for the electric light and shop engines when the main boilers are not in use.

The two suction pipes run forward from the pump casing, separating gradually until 12 ft. apart at the bow. At this point a tight radial joint is provided, which connects the pipes in the hull with the pipes from the suction head, and admits of the vertical motion necessary for dredging at different depths. The suction can be lowered to a maximum depth of 15 ft. Outside the hull the two suctions are framed together rigidly and are handled as a single piece. The raising and lowering is accomplished by means of sheaves and tackle attached to a derrick frame located over the forward end of the suction well. A straining frame is provided so that when the suction is lowered to a suitable depth it can be locked; it thus relieves the tackle of the weight, and maintains the suction at a constant depth. Each part

TABLE No. 7.—Capacity and Efficiency Tests of United States Dredge *Epsilon*. March 14th to 17th, 1898.

Number of test.	Duration of test.	*Conditions.		Forward movement of dredge, per minute.	Main Engine and Pump.											Jet Pump.	
							I. H.-P.					Head.					Head.
							Starboard.		Port.								
		Depth of suction.	Depth of water.		Steam pressure, pounds per square inch.	Revolutions of main pump, per minute.	High-pressure cylinder.	Low-pressure cylinder.	High-pressure cylinder.	Low-pressure cylinder.	Total.	Suction, feet water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.	Revolutions per minute.	Discharge, feet of water.
	Seconds.	Feet.	Feet.	Feet.													
1	122.2	15.0	7.8	3.0	153	182	170.6	88.5	228.4	124.1	611.6	13.0	44.7	1.6	59.3	188	22.1
2	102.5	15.0	8.5	3.0	160	181	194.8	101.8	232.4	113.0	642.0	15.8	34.7	3.4	53.9	200	26.7
3	82.6	15.0	8.5	4.4	155	181	176.5	88.0	278.8	169.1	712.4	16.4	34.7	4.4	55.5	191	24.4
4	54.5	14.0	7.3	5.8	155	182	233.0	136.2	260.7	145.8	775.7	18.1	37.7	3.8	59.6	224	31.3
5	61.5	14.0	7.0	9.4	165	182	231.7	137.1	278.7	153.6	801.1	19.8	37.7	4.1	61.6	232	32.5
6	66.4	14.0	8.0	7.1	160	181	239.0	139.8	239.6	119.9	738.3	17.0	37.7	4.3	59.0	228	32.5
7	57.5	15.0	9.0	.....	162	182	242.6	137.1	229.8	107.6	717.1	14.7	36.7	5.2	56.6	140	35.6
Average, excluding tests Nos. 1 and 2	64.5	14.4	7.96	†6.67	159.4	181.6	224.56	127.64	257.52	139.20	748.92	17.2	36.9	4.36	58.46	203.0	31.26

* Suction pipes: Two 24 ins. in diameter; length from mouth to pump = 69 ft. Discharge pipe: diameter, 32 ins.; length from stern of dredge to valve in barge = 1 036 ft.; length from pump to valve in barge = 1121 ft. † Average of Nos. 3 to 6.

TABLE NO. 7—(*Continued*).

Number of test.	MEASURING BARGE. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.).										EFFICIENCY.				Remarks.
	Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Total quantity of material pumped.	Quantity pumped, per second.	Velocity of discharge, per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand	Sand pumped, rate per hour.	Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency, per centum.	Sand pumped per hour, per I. H.-P.	
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu. yds.	Ft.-lbs.	Ft.-lbs.		Cu. yds.	
1	0.600	3.333	2.733	6 992.9	57.22	10.2	0.274	693.9	9.9	757.1	212 072	336 380	63.0	1.24	Mud lumps in barge, not average material.
2	0.417	3.700	3.283	8 400.2	81.95	14.7	0.217	549.6	6.5	714.9	276 069	353 100	78.2	1.11	Mud lumps in barge, not average material.
3	0.433	3.483	3.050	7 804.0	94.48	16.9	0.419	1 061.1	13.6	1 712.8	327 728	391 820	83.6	2.40	Medium fine sand.
4	0.750	2.617	1.867	4 777.1	87.65	15.7	0.464	1 175.1	24.6	2 874.9	326 496	426 635	76.5	3.71	Medium fine sand.
5	0.300	2.467	2.167	5 544.7	90.16	16.4	0.565	1 430.9	25.8	3 102.2	347 116	440 605	78.8	3.87	Medium fine sand.
6	0.500	2.900	2.400	6 140.9	92.48	16.6	0.555	1 405.5	22.9	2 822.3	341 020	406 065	84.0	3.82	Medium fine sand.
7	0.367	2.667	2.300	5 885.0	102.35	18.3	0.384	972.5	16.5	2 255.1	362 063	394 405	91.8	3.14	Medium fine sand.
Average, excluding tests Nos. 1 and 2.	0.470	2.8268	2.3568	6 030.34	93.424	16.78	0.4774	1 209.02	20.68	2 553.46	340 884.6	411 906.0	82.9	3.388	

TABLE No. 8.—Capacity and Efficiency Tests of United States Dredge *Zeta*. March 21st to 24th, 1898.

Number of test.	Duration of test.	* Conditions.		Forward movement of dredge per minute.	Revolutions of cutter engine per minute.	Main Engine and Pump.										
		Depth of suction.	Depth of water.			Steam pressure, pounds per square inch.	Revolutions of main pump per minute.	I. H.-P.					Head.			
								Starboard.		Port.		Total.	Suction, feet of water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.
								High-pressure cylinder.	Low-pressure cylinder.	High-pressure cylinder.	Low-pressure cylinder.					
	Secs.	Feet.	Feet.	Feet.												
1	62.0	14	7.5	4.8	120	145	182	224.5	137.1	197.2	159.6	718.4	13.0	34.7	4.5	52 2
2	60.0	13	7.5	6.0	186	150	182	218.3	120.6	197.5	152.7	689.1	12.4	35.7	3.3	51.4
3	69.6	13	6.0	4.0	155	146	182	225.2	122.3	189.3	141.4	678.2	10.2	35.7	4.2	50.1
4	70.0	14	8.0	3.2	183	146	181	215.4	119.1	186.7	139.8	661.0	10.7	35.7	4.9	51.3
5	61.6	14	8.5	5.0	180	144	180	222.7	129.6	191.5	144.2	688.0	11.3	33.7	4.2	49.2
Average	64.64	13.6	7.5	4.6	164.8	146.2	181.4	221.22	125.74	192.44	147.54	686.94	11.52	35.1	4.22	50.84

* Suction pipes: Two, 24 ins. in diameter; length from mouth to pump = 69 ft. Discharge pipe: Diameter, 32 ins.; length from stern of dredge to valve in barge = 1 036 ft.; length from pump to valve in barge = 1 121 ft.

TABLE NO. 8—(*Continued*).

Number of test.	Measuring Barge. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.)										Efficiency.				Remarks.
	Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Total quantity of material pumped.	Quantity pumped per second.	Velocity of discharge per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency. per centum.	Sand pumped per hour, per I. H.-P.	
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu yds.	Ft.-lbs.	Ft.-lbs.		Cu.yds.	
1.....	0.417	2.717	2.300	5 885.0	94.92	17.0	0.345	873.7	14.8	1 878.9	309 677	395 120	78.4	2.62	Medium fine sand. Sheared four pins before test was obtained.
2.....	0.650	2.567	1.917	4 905.0	81.75	14.6	0.231	585.0	11.9	1 300.0	262 622	379 005	69.4	1.89	Medium fine sand. Test made after shearing four pins, only one agitator running.
3.....	0.317	2.817	2.500	6 396.8	91.91	16.5	0.274	693.9	10.8	1 329.4	287 793	373 010	77.2	1.96	Medium fine sand. Test made after shearing two pins.
4.....	0.433	3.150	2.717	6 952.0	99.31	17.8	0.222	562.2	8.1	1 070.9	318 413	363 550	87.6	1.62	Medium fine sand. Test made after shearing five pins and with only one agitator running.
5.....	0.433	2.650	2.217	5 672.6	92.09	16.5	0.227	574.9	10.1	1 244.4	283 177	378 400	74.8	1.81	Medium fine sand. No pins sheared.
Average...	0.450	2.7802	2.3302	5 962.3	91.996	16.48	0.2598	657.94	11.14	1 364.72	292 336	377 817	77.5	1.98	

of the suction head is 10 ft. long, making the suction 20 ft. wide over all.

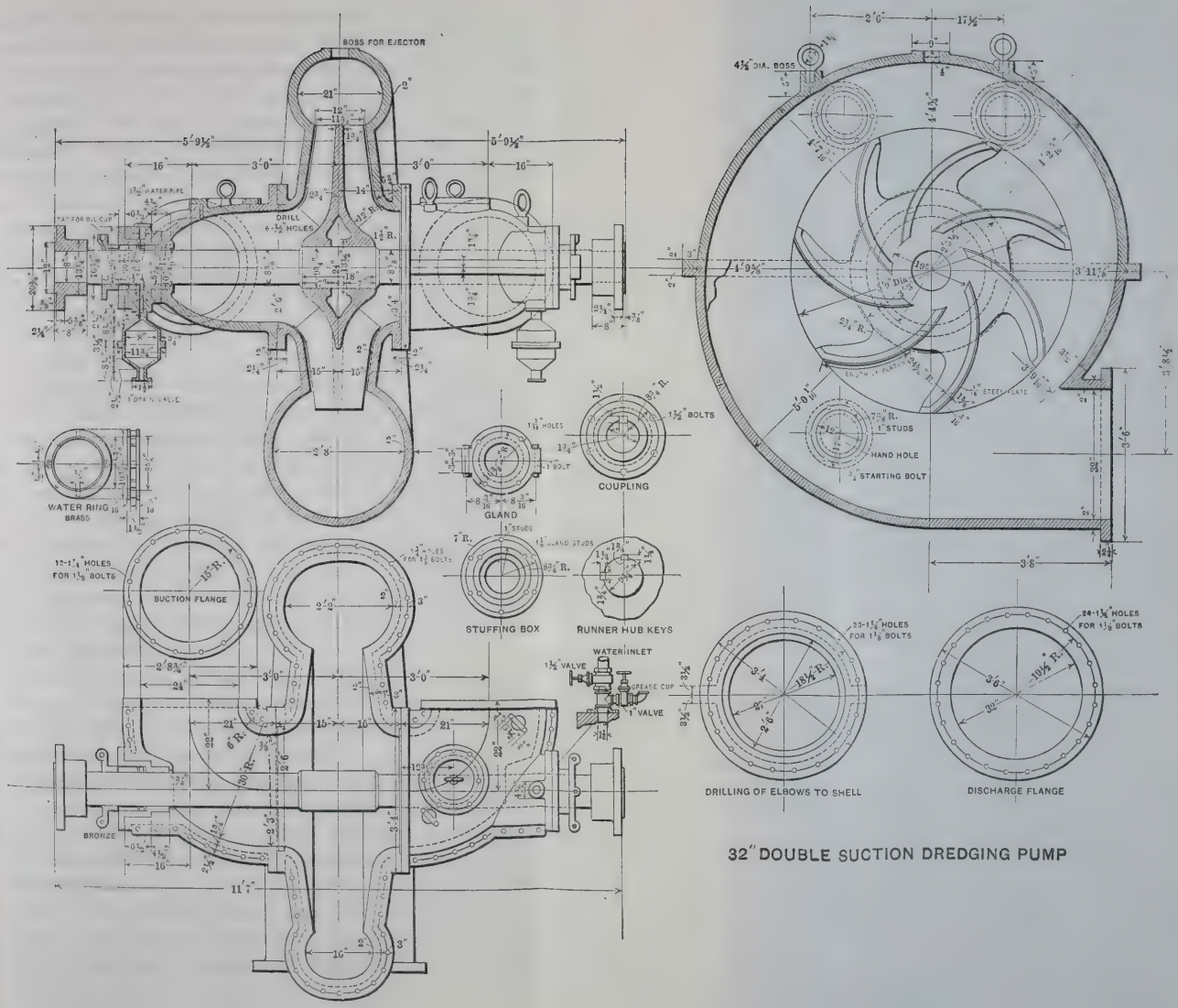
The *Epsilon* suction head has a pressure chamber on the under side at the front end connecting with the jet pump. Ten 3-in. jet nozzles are screwed into the front side of this pressure chamber and serve to loosen and stir up the sand and induce the flow of material to the suction.

The discharge pipe passes straight aft from the pump between the two central keelsons, and rests on the floor beams for a distance of about 52 ft. From this point it rises gradually for a distance of 22 ft. until the center of the pipe is 4 ft. above the bottom of the boat. Then it runs horizontally straight out through the stern, projecting far enough to couple on the floating pipe line. The pipe is flattened out near the pump casing so as to depress it below the water-line and thus facilitate priming, which is done by using a steam syphon at the top of the casing. The floating discharge pipe is similar to those previously described, except that greater buoyancy is provided, and the shape of the float is somewhat flatter. Fig. 2, Plate XXVIII, shows some of these pipes under construction and gives a good idea of their form. They are built in 50-ft. lengths.

Traveling cranes are provided for the easy handling of the machinery of the engine room and the hoists; and there is one steam capstan at each corner of the dredge. Steam syphons are provided for all the water-tight compartments. Six 10-in. and ten 7-in. mooring piles of the usual form are provided for each dredge.

A velocimeter is provided for showing the velocity of flow in the discharge pipe. This consists of two vertical tubes which pierce the discharge pipe. They are set in the vertical plane passing through the longitudinal axis of the discharge pipe. About 5 ins. below the upper inner surface of the pipe, ¼-in. tubes, 6 ins. long, are inserted horizontally; the one nearest the pump being open toward the pump to receive the pressure of flow and the other turned in the opposite direction. The upper ends of the vertical tubes connect with glass tubes attached to suitable scales. The difference in the height of the columns of water in the two tubes, when water is flowing through the discharge pipe, gives results from which the velocity of flow can be deduced.

Weighing apparatus, designed to show the percentage of solid matter passing through the discharge pipe, is also provided. To ac-



32" DOUBLE SUCTION DREDGING PUMP

complish this, a suitable length of the pipe is connected at each end by rubber thimbles, which leaves the joints flexible. One end of this pipe is then supported by a weighing apparatus properly counter-balanced. An indicator is located where it can be readily seen by the engineer. The pointer is set so that it reads zero on the dial when water only is pumped. When sand is pumped the pointer will show on the dial the weight of the same at any moment, and consequently the percentage of solid matter.

Both dredges have cabins, for the protection of the machinery and to provide quarters for the crew. On the main deck the house is 34 ft. wide, 113 ft. long and 12 ft. high. Above this is the cabin proper, 32 ft. wide, 109½ ft. long and 9 ft. high, built and arranged as in ordinary steamboat practice. There are ample accommodations for a crew of fifty.

The weights of the various parts of each of these dredges are about as follows: Hull, 442 388 lbs.; main pump and engines, 117 060 lbs.; cutter engine and agitator, *Zeta*, 25 000 lbs.; jet pump and engine *Epsilon*, 19 010 lbs.; winding engines and machinery, 33 436 lbs.; fire and feed pumps, 8 100 lbs.; heater, 4 925 lbs.; electric plant, 4 200 lbs.; derrick, A-frame, travelers, sheaves, pipes and fittings, deck fittings, capstans, etc., 37 500 lbs.; discharge and suction pipes pertaining to hull, *Epsilon*, 54 447 lbs.; same for *Zeta*, 51 509 lbs.; boilers, stacks, etc., complete, 195 000 lbs.; cabin, complete, 120 000 lbs.; total approximate weight of each dredge, 1 057 118 lbs.; weight of floating pipe line, 325 000 lbs.

The efficiency tests of these dredges were begun about the middle of January. Owing to delays occasioned by high water and some minor modifications that were required, the capacity tests were delayed until the latter part of March, 1898.

The results of these tests are given in Tables Nos. 7 and 8.

The specifications and plans of all the dredges of the Mississippi River Commission have been prepared under the direction and subject to the approval of the Committee on Dredges, consisting of Major Thomas H. Handbury, Corps of Engineers, U. S. A.; Henry Flad, M. Am. Soc. C. E., and B. M. Harrod, Past-President, Am. Soc. C. E.

The dredges *Gamma*, *Delta*, *Epsilon* and *Zeta* were constructed under the direction of Captain H. E. Waterman, Corps of Engineers, U. S. A.

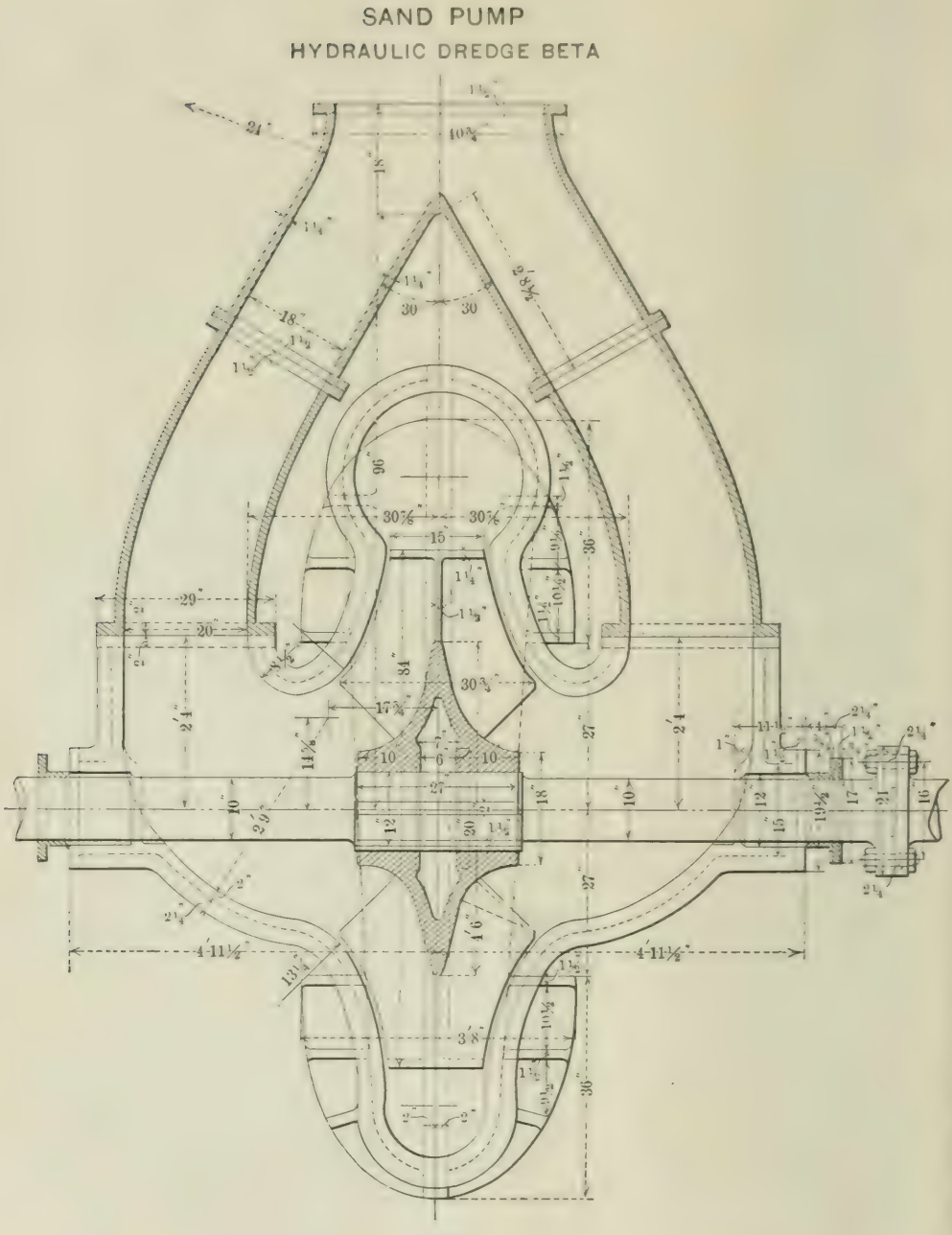


FIG. 11.

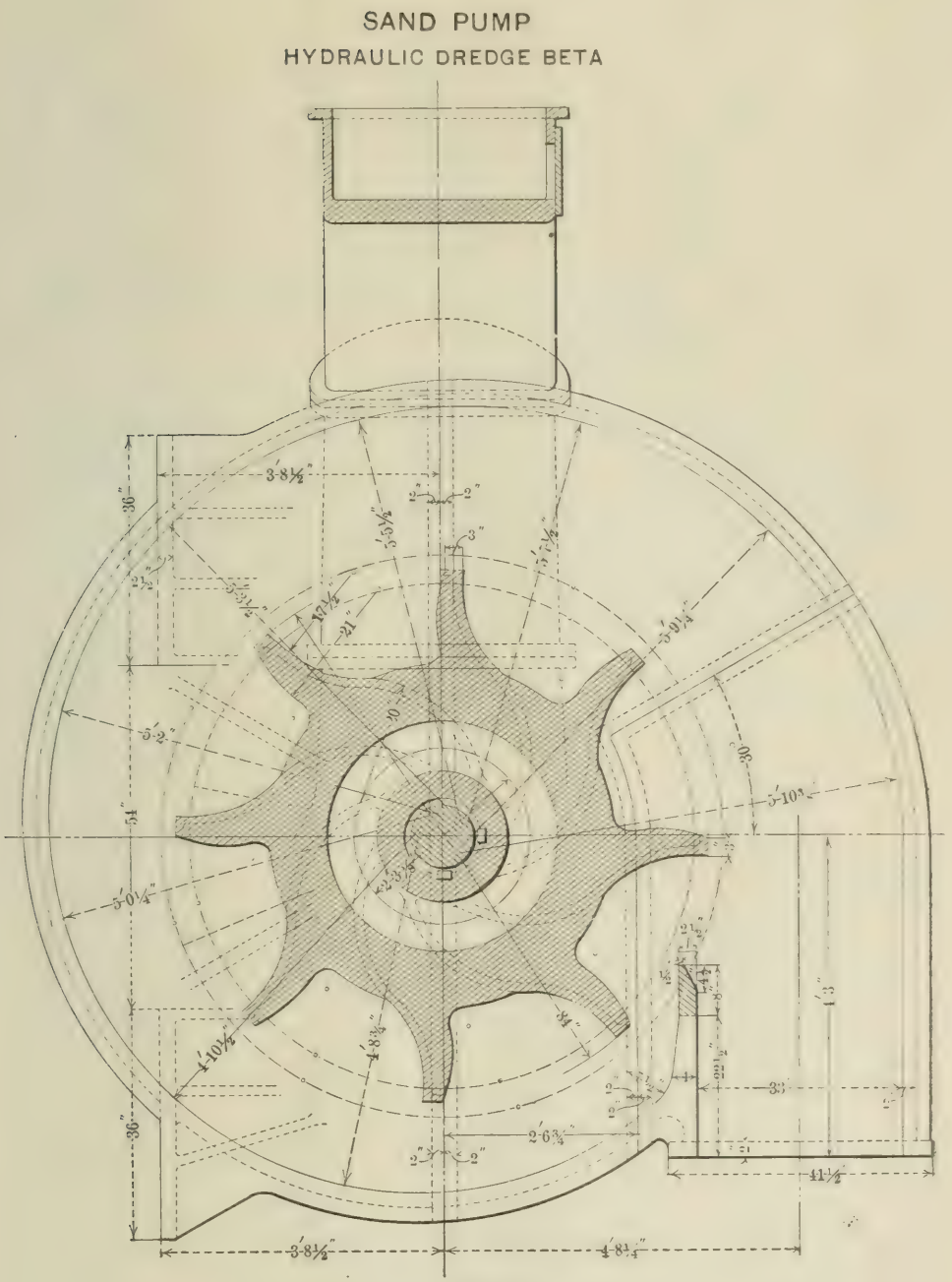

SAND PUMP
HYDRAULIC DREDGE BETA
3'-8½"
2"
2"
3"
5'-5¾"
5'-7½"
5'-9¼"
5'-10"
5'-2"
5'-0¼"
4'-10½"
4'-8¾"
4'-8¼"
2'-6¾"
41½"
36"
30"
51"
21"
3'-8½"
4'-8¼"

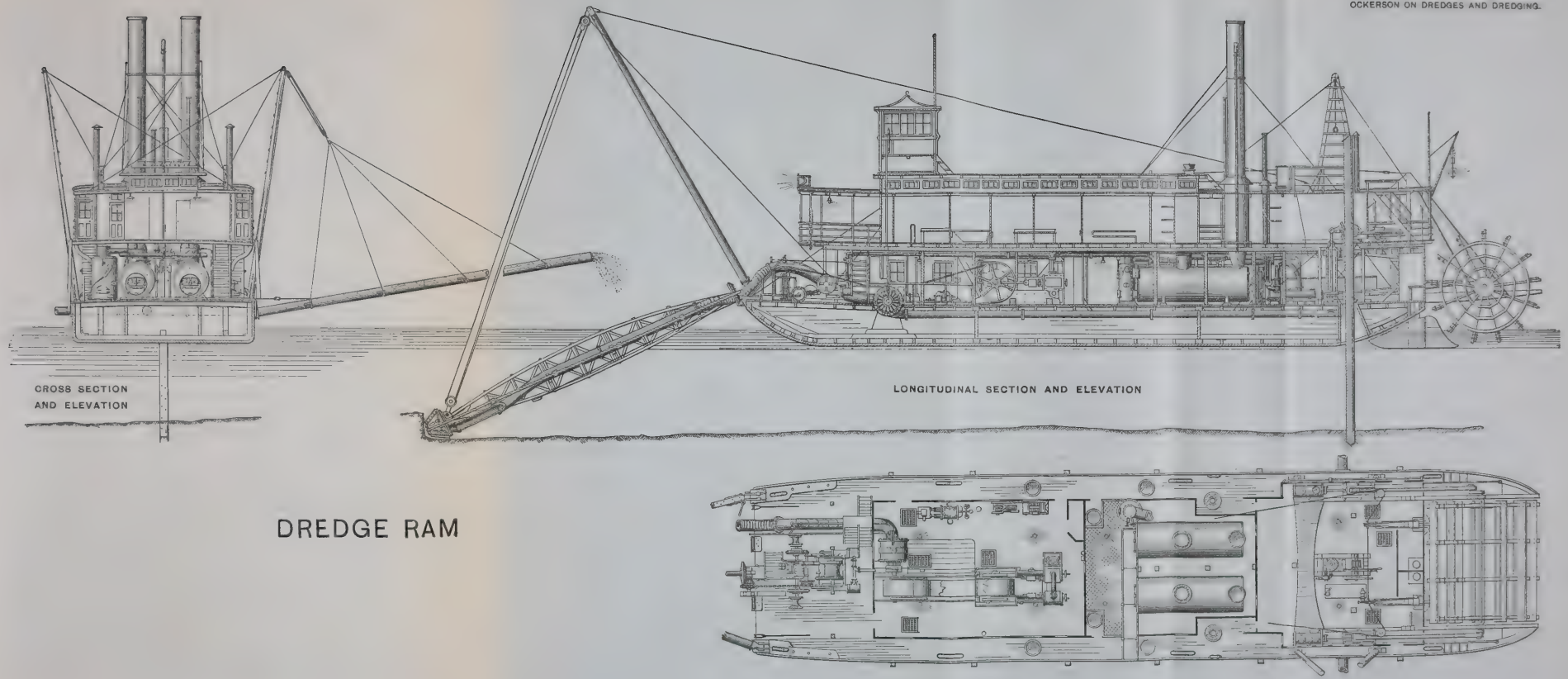
FIG. 12.

TABLE No. 9.—Principal Features of U. S. Dredge Boats Operating on the Mississippi River below Cairo, Ill.

Name of Dredge.	*Alpha.*	*Beta.*	*Gamma.*	*Delta.*	*Epsilon.*	*Zeta*
When built	1893.	1895.	1897.	1897.	1897-98.	1897-98.
Cost	$87 000	$217 000	$86 000	$125 000	$102 000	$106 000
Capacity in cubic yards of sand per hour.	600	2 000	800	1 000	1 000	1 000
Hull—Kind	Wood.	Steel.	Steel.	Steel.	Steel.	Steel.
Size	140′ x 36′ x 8′	172′ x 40′ x 7′ 2″	138′ x 38′ x 8′	175′ x 38′ x 8′ 4″	157′ x 40′ x 7′ 9″	157′ x 40′ x 7′ 9″
Working draft	4′ 10″	6′ 6″	4′ 3″	4′ 3″	4′	4′
Main engines—Number	1	2	1	1	2 coupled	2 coupled
Kind	Cd. non-cond.	4 cyl. trpl. exp. 2 crank cond.	Horiz. cross comp. cond.	Vertical comp. cond.	Horiz. tandem coup. non-cond.	Horiz. tandem coup. non-cond.
Size	15″-27″ x 20″	20½″-33″-38″ -38″ x 24″	18″-22½″ x 22″	24″ 48″ x 24″	16″-26″ x 18″	16″-26″ x 18″
Horse-power	300	2 000	500	800	650	650
Revolutions per minute	140	135	150	150	180	180
Kind of condenser	None.	Jet.	Jet.	Jet.	None.	None.
Main dredging pumps—Centrifugal	1	2	1	1	1	1
Diameter of runner	6′	7′	5′ 9″	7′ x 28″ wide.	5′ 9″	5′ 9″
Kind of suction	Side.	Divided.	Divided.	Side.	Divided.	Divided.
Diameter of suction	30″	Six 19½-in.	Two 24-in.	Four 17½-in.	Two 24-in.	Two 24-in.
Discharge pipes—Number	1	2	1	1	1	1
Discharge pipes—Size	30″	33″	34″	34″	32′	32″
Suction head in feet of water	7	18	9	18	12	12
Delivery head in feet of water	20	29	37	45	28	28
Velocity—Feet per second	10	14	10	15	13	13
Sand agitators	6 jets 2½″ diam.	6 vert. cutters.	9 jets 2½″ diam.	Horiz. cutters.	10 jets 3″ diam.	Oscillating rake.
Engines, kind	Cd. non-cond.	Cross cd. non-cond.	Comp. cond.	2 simple.	Vert. comp. non-cond.	2 simple.
Engines, size	.....	14½″-29″ x 18″	12″-22″ x 14″	12¼″ x 15″	12″-22″ x 12″	Each 12″ x 16″
Engines, revolutions per minute	190	120	160	.....	200	180
Cutters, revolutions per minute	.....	12	.....	5	.....	60 double strokes
Engines, horse-power	125	.....	200	.....	150	80
Jet pumps	Centrifugal.	.....	Centrifugal	.....	Centrifugal.	.....
Suction and discharge diameter	10″	.....	18″	.....	18″ and 15″	.....
Suction head	6	.....	7	.....	5	.....
Delivery head	20′	.....	28′	.....	30′	.....
Boilers—Number	4	4	6	4	6	6
Kind	Miss. River.	Heine 375 H.-P.	Miss. River.	Heine 250 H.-P.	Miss. River.	Miss. River.
Size	42″ x 28′	.....	48″ x 28′ Five 11″ flues.	.....	48″ x 28′ Three 11″ & Two 13″ flues.	48″ x 28′ Three 11″ & Two 13″ flues.
Pressure used	135 lbs.	165 lbs.	140 lbs.	155 lbs.	150 lbs.	150 lbs.
Coal used per 24 hours	500 bu.	2 088 bu.	400 bu.	1 200 bu.	.....	.....
Total cost of running dredge per day	$98.70	$221.63	$100.51	$111.76	.....	.....

Cost of steam tender per day, $37. Cost of pile sinker per day, $13.

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XL, No. 838.
OCKERSON ON DREDGES AND DREDGING.



DREDGE RAM

Another dredge has been designed for work on the river below Cairo, and the contract for its construction has been let. This will be a single-pump dredge, similar in form to the *Epsilon*, but will be provided with side wheels and propelling machinery. The contract price for this dredge is $94 970.

Perhaps the most important feature of a hydraulic dredge is its pump. As there is a wide variance in practice as to the form and size of pump runners, it will probably be interesting to show the details of the several pumps now in use on the Mississippi River. These are shown in the following plates and figures: *Alpha*, Plate XXIX; *Beta*, Figs. 11 and 12; *Gamma*, Plate XXX; *Delta*, Plate XXXI; *Epsilon* and *Zeta*, Plate XXXII.

The general features of the dredges now in use on the Mississippi River below Cairo are summarized in Table No. 9.

Two dredges for use on the Mississippi River between the Missouri and Ohio Rivers are now under construction and are practically completed. They were designed and constructed under the direction of Major Thomas H. Handbury. They will be ready for use during the next low-water season. It is expected that a navigable channel 6 ft. deep can be maintained with these dredges, aided by the temporary jetties and the jet dredge.

The hulls of these dredges are of steel, 160 ft. long and 40 ft. wide, with 6½ ft. depth of hold. Their working draft will be about 3½ ft. The framing of the hulls consists of 12-in. channels weighing 25 lbs. per foot for floor beams; side frames 3 x 4-in. angles, 11 lbs. per foot, riveted to web of floor beams. The above frames are spaced 12 ins. in the engine and boiler pit, and 30 ins. through the remainder of the hull. There are 10 keelsons, running the entire length of the boat, made of 15-in. steel I-beams. The outboard beams weigh 55 lbs. per foot and the intermediates 45 lbs. per foot. Each keelson is riveted to each floor beam. The center strake of the bottom plating is $\frac{3}{8}$ in. thick, the other strakes being $\frac{5}{16}$ in. thick. The deck plating is $\frac{1}{4}$ in. thick. The upper strake of the side plating is $\frac{3}{8}$ in. thick, and the other side plates are $\frac{5}{16}$ in. thick. The hull is divided into 11 water-tight compartments.

Each dredge is provided with two centrifugal pumps with a single suction in the axis of the pump discharging from the lower side of the casing. The suction is 20 ins. in diameter and the discharge at the

pump is the same diameter, but is expanded to 24 ins. at the discharge pipe. The casing of the pump is 48 ins. in diameter and 28 ins. across the inside. It is provided with liners which can be renewed when worn. The pump runner is 48 ins. in diameter and is provided with three reversed-curve arms. The ends of these arms are fitted with detachable extension plates made of mild steel. An adjustable thrust bearing is provided. A water-jet is connected with the main bearing to keep out the sand and grit when the pump is running.

Each pump is operated by a direct-connected, horizontal, compound, non-condensing engine. These engines are designed to run 200 revolutions per minute and develop 300 H.-P. at a boiler pressure of 140 lbs. This speed is to be regarded as the average work of the engine, and is expected to maintain through the suction and discharge pipes the velocity necessary to carry at least 20% of sand and 80% of water under a maximum head of 30 ft.

A jet pump is provided with each main pump which serves to loosen up the material at the end of the suction and thus facilitate its passage into the suction pipe. The jet pump has two high-pressure cylinders 8 ins. in diameter, two low-pressure cylinders 12 ins. in diameter, and two water plungers 12 ins. in diameter; all having 10-in. stroke. The capacity of the pump is 1 200 galls. per minute against a water pressure of 65 lbs. when running compound under a steam pressure of 150 lbs. These pumps discharge through 8-in. pipes terminating in three bronze nozzles 1½ ins. in diameter, their extremities radiating to a distance of about 30 ins. apart and just beneath the suction of the main pump.

Steam is supplied by six boilers of the Mississippi River steamboat type. These boilers are 48 ins. in diameter and 28 ft. long, and each shell contains five 11-in. flues. They are made of marine steel of 60 000 lbs. tensile strength, and in all respects conform to the rules of the Board of Supervising Inspectors. They are designed for a working pressure of 140 lbs. per square inch. The boilers are arranged in two batteries, with three boilers in each, which are connected by one 30-in. steam drum and two 15-in. mud drums. The steam connections are so arranged that one or both batteries can be used at will. There are two smokestacks 42 ins. in diameter and 70 ft. high. The boilers are supplied by two feed pumps with outside-packed plungers 4½ ins. in diameter and 10-in. stroke.

FIG. 1.

FIG. 2.

The boilers and machinery rest on I-beam keelsons, the hull being left open for the space occupied by these parts. The remainder of the hull is decked over with steel plates. A vertical anchor spud is placed near the bow of the hull.

The main suction pipes are suspended from a derrick frame and are raised or lowered as required by winches, so that each suction can be operated independently. The suction head opening, or end of the suction pipe, is 20 ins. high by 120 ins. long. The inner end of the suction works on a hinge and slip joint.

There is one 24-in. tank-steel discharge pipe for each pump. Each of these pipes is attached to the fixed sections in the hull by a horizontal swivel joint which admits of swinging the pipe out sidewise. The pipes are in lengths of 24 ft., and each length is floated on a steel pontoon with pointed ends, the pipe being attached to the float at the middle joint in such a way that the pontoons lie parallel with the current when the pipes are deflected to one side. Each discharge pipe is 500 ft. long, the several joints being connected together with rubber gaskets. The first joint is connected to the fixed joint at the stern of the dredge by a heavy rubber coupling 3 ft. 8 ins. long.

In operating the dredge, two iron piles are set on line with the cut, and the dredge is attached to them by two $\frac{3}{4}$-in. wire cables 1 200 ft. long. The dredge end of each of these cables is attached to a winch having a 42-in. drum. These winches have double engines and are geared so as to give speeds from $\frac{1}{4}$ in. to 7 ins. per second. The dredge is pulled ahead as rapidly as the dredged material can be taken through the pipes, which, of course, varies with the depth of material handled.

The electric light plant consists of one 4 000 candle-power search light, two 2 000-candle-power arc lights, and seventy-five 16-candle-power incandescent lights. The dynamo is a 110-volt machine of the Western Electric pattern, operated by a horizontal, single expansion 7 x 10-in. engine.

A cabin provides ample quarters for officers and crew, and a machine shop gives facilities for repairs.

Dredge Ram.

In the fall of 1893 the dredge *Ram* was completed by the Bucyrus Steam Shovel and Dredge Company, for use in keeping open the channel from the Mississippi River into the Red and Atchafalaya Rivers.

It was built under the direction of Captain John Millis, Corps of Engineers, U. S. A. This dredge cost $69 500. The capacity is about 300 cu. yds. of mud per hour, delivered through 300 ft. of pipe. It is provided with propelling machinery such as commonly used on stern-wheel steamboats. The hull is 95 ft. long, 27 ft. beam and 7½ ft. deep. It is built of long-leaf yellow pine. It has two longitudinal and two transverse bulkheads, dividing the hull into nine water-tight compartments. The dredging pump is an Edwards centrifugal, with 15-in. suction and discharge. The intake is from the starboard side and the discharge from the bottom of the casing. The pump is run through a belt connection by a horizontal, compound-condensing engine. The cylinders are 14½ and 26 ins. in diameter, with a stroke of 20 ins.

This dredge can work in any depth up to 30 ft. It is so arranged that material can be discharged on either side at will. A vertical anchor spud is provided in the stern of the boat, on which it swings by means of kedge anchors laid out from either side of the bow and maneuvered by winding drums. In this way a wide cut can be made by swinging the cutters from side to side.

The cutter and suction is supported by an A-frame, the inner legs of which are pivoted to the sides of the bow. The frame is raised or lowered with wire rope tackle attached at the upper end to suitable shears and operated by winding drums. The cutter is a conical steel casting with eight steel blades forming a cutter 24 ins. in diameter at the end and 54 ins. in diameter at the base. This cutter is at the outer extremity of a 5-in. steel shaft, which is revolved by gear wheels at the bow of the boat, and operated by a sprocket wheel and chain. The suction proper starts at the base of the cutter.

The propelling engines have 15-in. cylinders and a 52-in. stroke. There is a combined, duplex air and feed pump and condenser, with 7½-in. steam cylinder, 7-in. air cylinder and 10-in. stroke. Fire and bilge pumps are also provided.

The cutter shaft and winding drums are operated by a pair of auxiliary engines, the cylinders of which are 9 ins. in diameter and 12-in. stroke. The drums are worked by separate clutches.

Steam is supplied by two main boilers 16 ft. long and 60 ins. in diameter. Each shell has 74 3-in. tubes and a corrugated furnace flue 40 ins. in diameter and 72 ins. long.

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
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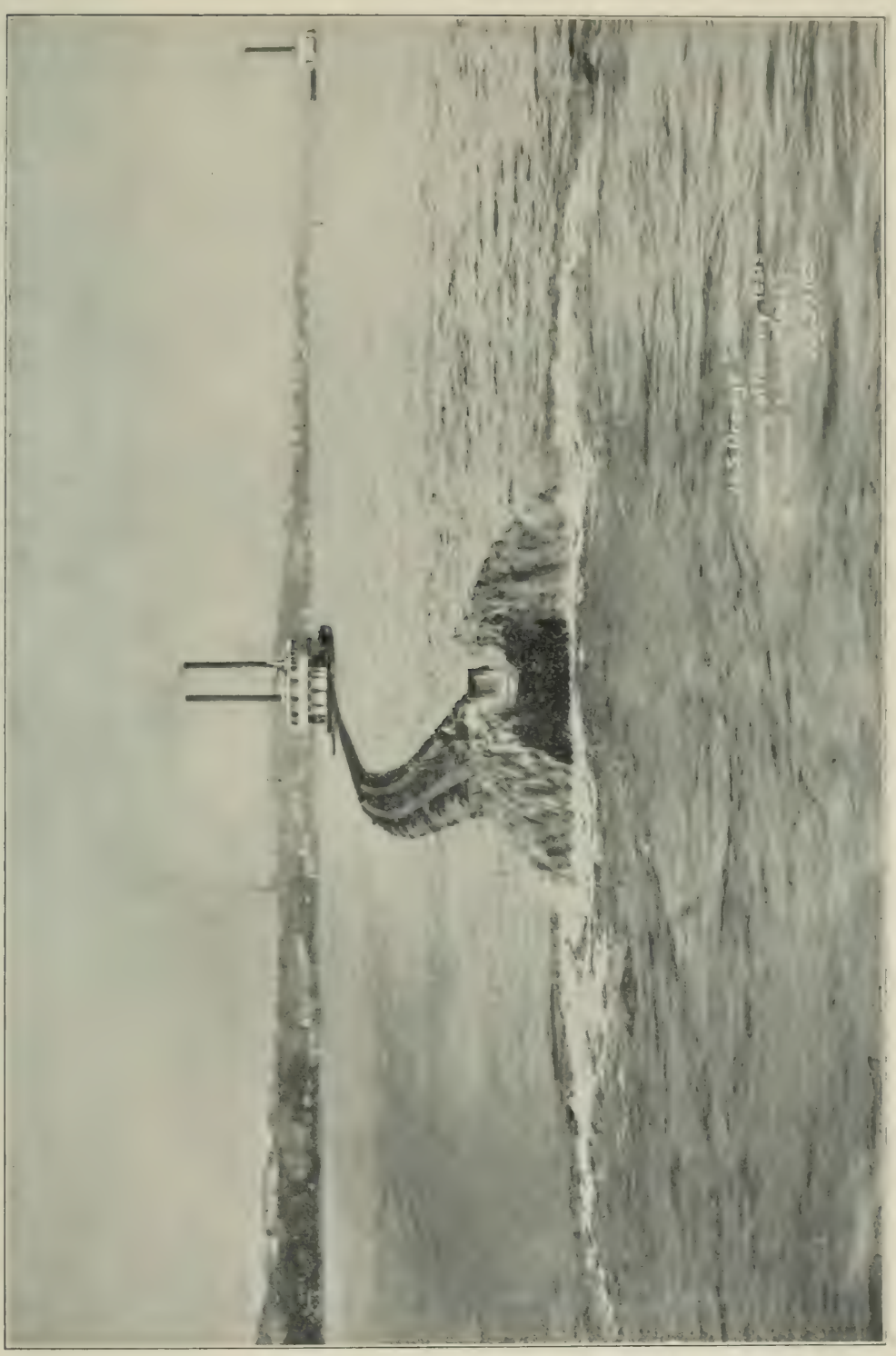
OCKERSON ON DREDGES AND DREDGING.



The boat is lighted throughout by electricity. There is a double-deck cabin which provides ample accommodations for the crew and protects the machinery.

This boat has been quite successful from the start. During the low-water season of 1894 she dredged 265 000 cu. yds. of material at a cost of a trifle over 4½ cents per yard. The material was largely mud and clay. The material is generally delivered through about 50 ft. of pipe suspended from a derrick and attached to the boat, as shown in Plate XXXIII, from which a good general idea of this boat can be obtained.

It has also been used with considerable success in building levees. It is estimated that under favorable conditions a levee 10 ft. high can be built at a cost of about 6 cents per cubic yard.

Menge Dredges.

In 1888 two dredges of the elevator type were procured, one for use in Vicksburg harbor and one for the mouth of the Red River. These were called the *Menge* and the *Pah-Ute*.

In actual work the *Menge* has dredged 4 000 cu. yds. in 10 working hours. This dredge cost about $18 000. The *Pah-Ute* has about half the above capacity and cost $10 000. These dredges, as generally equipped, deliver the dredged material into dump scows which are towed to the desired point and dumped. In some favorable situations the material is delivered through sluice boxes supported on barges. These dredges, if properly constructed, are very successful when operated in soft material.

At the mouth of the Red River the efficiency of propeller wheels and stern-wheel boats has been thoroughly tried in removing the sediment and keeping the channel open. Under favorable conditions some good results have been obtained in this way; but on the whole, it is generally conceded to be a very poor makeshift.

Dredge *Bayley*.

The dredge *G. W. R. Bayley* was built by Carroll & Company, of Pittsburg, for use in South Pass. It arrived at Port Eads in the fall of 1877. This boat is constructed of iron throughout. The hull is about 200 ft. long over all, 32 ft. wide, and 10 ft. depth of hold. With fuel on board, it draws about 5 ft. of water. It is self-propelling

by means of side wheels 25 ft. in diameter, with buckets 11 ft. long. These wheels are operated by two engines with 21-in. cylinders and 7-ft. stroke. The boat is provided with a rudder at each end to obviate the necessity of turning when dumping or returning to the cut.

This dredge has a centrifugal pump of the Andrews Cataract pattern, with a runner 6 ft. in diameter and 3 ft. wide, mounted on an 8-in. steel shaft. It is operated by two independent condensing engines, placed forward of the pump, having 24-in. cylinders and 20-in. stroke. The wrought-iron suction pipe is 27 ins. in diameter. This suction operates in a recess 4 ft. wide, and 25 ft. in length, located at the stern of the boat. It connects with the pipe in the boat, just above the bottom plating and near the pump, by means of a joint which admits of a deflection of 30° from the axis in any direction. The lower end of the suction is curved and flattened out to a width of 4 ft., and is armed with a steel scraper. The suction is supported on a flat plate resting on the sand to prevent the scraper from cutting too deep. This scraper takes in a slice of sand about 8 ins. deep and 4 ft. wide. A stream of water of about half the above depth and the same width enters the suction at the same time.

The movable part of the suction, when in operation in 26 ft. of water, stands at an angle of about 35 degrees. When not in use it is lifted above water, under the extension of the main deck, the end projecting about 15 ft. aft of the stern post. The discharge pipe is 30 ins. in diameter and is arranged so that the spoil can be deposited in the spoil bins on board, or delivered on either side of the boat through pipes swung from derricks. There are four boilers, 42 ins. in diameter and 26 ft. long, with four flues in each.

The four spoil bins stand forward of the engines. They are 60 ft. long, 19 ft. wide and about 20 ft. deep, and have a capacity of about 512 cu. yds. of dredged material. These tanks are filled in about 7 minutes, when the suction is raised and the boat proceeds to the dumping ground. Two-thirds of the available time is occupied in unloading. Overflow gates are provided so as to allow the water to escape while the sand settles to the bottom, thus insuring a greater load of solid matter. In a working day of 13 hours the average amount of solid matter removed is 1 309 cu. yds. The tanks are divided into compartments, each of which terminates in a hopper having a 4-ft. opening through the bottom of the hull. These openings are

LOWER POINT PLEASANT BAR

79.5 MILES BELOW CAIRO

SCALE OF FEET

1000 0 1 2 3000

ALL DEPTHS REFER TO THE LOWEST STAGE OF 1896.

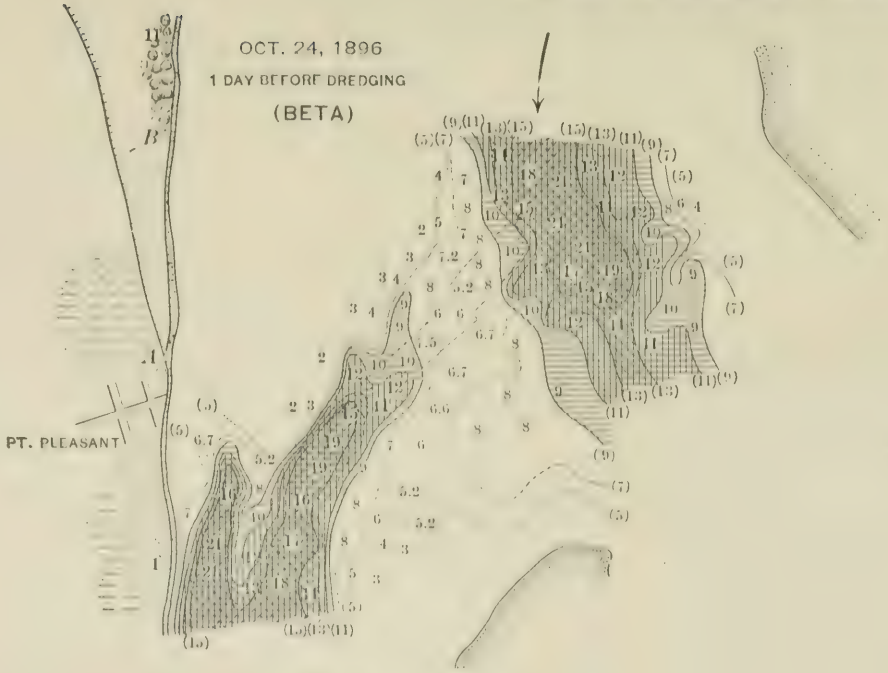


FIG. 13.

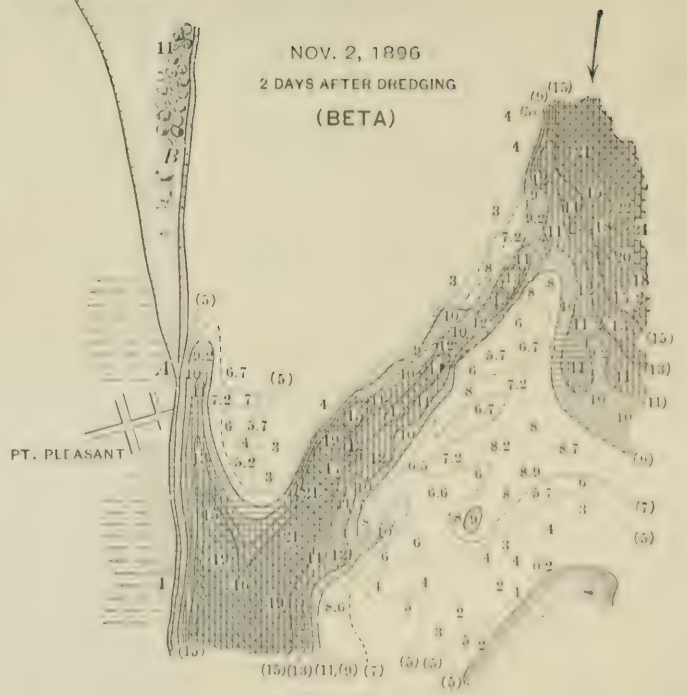


FIG. 14.

TABLE No. 10.—Summary of Dredging Operations, Mississippi

OF

Place and date.	Name of dredge.	Cuts.				Average depth in feet.			
		Number.	Total length. Feet.	Time required. Hours.	Rate. ft. per hour.	Suction.	Ahead of dredge.	Astern of dredge.	Material removed.
Island No. 40 (211 L), August 31st to September 24th, 1896....	*Beta.*	10	5 945	121.0	49.1	15.3	9.2	13.4	4.2
Island No. 34 (179 R), September 25th to 30th, 1896.....	*Beta.*	3	2 265	66.0	34 3	15.6	9.5	13.5	4.0
Osceola Bar (164.5 R), September 30th to October 9th, 1896....	*Beta.*	9	5 660	63.7	89.0	15.0	9.4	13.5	4.1
Hathaways Lower Crossing (102 L), October 9th to 15th, 1896	*Beta.*	16	12 175	87.0	140.0	20.5	14.8	18.7	3.8
Cherokee Crossing (89 R), second time, October 15th to 24th, 1896.....	*Beta.*	11	6 435	78.2	82.3	17.0	10.8	15.8	5.0
Lower Point Pleasant, Mo. (79.5 R), October 25th to November 3d, 1896	*Beta.*	13	9 415	109.2	86.2	16.0	7.7	13.5	5.8
Cherokee Crossing (89 R), third time, November 4th to 13th, 1896.....	*Beta.*	10	8 980	175.0	51.3	17.3	10.7	16.1	5.4
Cherokee Crossing (89 R), first time, September 5th to 28th, 1896.....	*Alpha.*	6	4 610	294.0	15.7	12.0	7.8	10.9	3.0
Upper Point Pleasant, Mo. (79 R), September 29th to October 23d, 1896....	*Alpha*	4	6 370	306.7	20.8	13.0	9.3	11.7	2.4
Compromise Bar (77 L). October 24th to November 15th, 1896....	*Alpha.*	7	5 405	257.7	21.0	12.7	9.9	11.6	1.7

closed by valves operated by hydraulic jacks with a capacity of about 12½ tons. The sand is washed out of the tanks by means of two jets in each hopper.

The capacity of the dredge when delivering the material as described is about 350 cu. yds. per hour. If the material is simply delivered overboard, the capacity is about 1 000 cu. yds. per hour. When dredging, the boat moves down stream, with the suction lowered, at the rate of about 2½ miles per hour.

This dredge cost about $150 000. It was designed by Jas. B. Eads, M. Am. Soc. C. E., for the special purpose of working in deep rough water with strong currents. * This boat is now over twenty years old, but is still serviceable.

The foregoing descriptions cover a fleet of eight hydraulic dredges of large capacity and five other dredges, all of which are now available

* See "The Mississippi Jetties," by E. L. Corthell, M. Am. Soc. C. E.

River Between Cairo and Memphis. During Low-Water Season 1896.

Heads in feet.		Revolutions. Ave. per min.		Steam pressure.	Distribution of time by hours.									Total cost.
Suction.	Delivery.	Pumps.	Cutters.		Placing plant.	Dredging.	Changing cuts.	Repairs.	Passing boats.	Making up tow.	Towing.	Sundays and holidays.	Total time.	
14.4	19.1	129	15	161	53.5	121.0	25.2	12.7	1.0	39.0	104.0	19.0	375.5	$6 122.51*
15.2	21.6	134	16	165	16.5	66.0	5.0	14.5	.....	6.5	25.5	.....	134.0	1 793.42
15.2	20.8	135	15	167	11.0	63.7	7.7	6.5	.....	20.5	97.5	10.0	217.0	2 379.05
15.0	23.4	134	16	160	23.0	87.0	.....	.....	.....	5.5	28.0	.....	143.5	1 671.59
14.5	24.5	135	16	163	9.5	78.2	.....	9.7	.....	18.0	24.0	.....	139.5	1 742.58
15.2	25.6	136	16	162	13.0	109.2	.....	62.7	.....	7.0	24.0	24.0	240.0	2 315.51
13.6	25.9	137	14	164	14.0	175.0	.....	24.0	.....	24.0	.....	3.0	240.0	3 008.93
5.2	16.8	124	Water jet.	138	32.0	294.0	11.0	143.0	.....	24.0	8.0	64.0	576.0	3 309.51†
6.3	19.5	118		141	14.2	306.7	15.7	79.2	.....	.....	9.0	70.0	495.0	3 456.16†
5.8	20.4	120		139	13.5	257.7	8.25	93.0	.....	37.5	10.5	59.5	480.0	3 473.68†

* Capsizing of pipes delayed the work 224½ hours.

† 114¾ hours. } Repairs due to breaking of runner of main pump.
78 hours. }
91 hours. }

for use in improving the low-water navigation of the Mississippi River. The dredge *Alpha* has been operated for three seasons, the *Beta* for two seasons, and the *Gamma* and *Delta* for one season. They have all demonstrated the practicability of moving economically immense quantities of material in a short space of time. They have also shown that good navigable channels can be opened through the reefs that obstruct navigation.

As far as the machines themselves are concerned, they seem to meet the requirements fully. The value of these dredges as aids to navigation depends far more on the proper location and direction of the cuts to be made than on the efficiency of the machinery. Experience may show that in some cases the dredges can be made more efficient by filling side chutes and thus throwing the water into

one channel, rather than attempting to enlarge one of the channels by dredging out the material. It may also be found desirable in some cases to discharge the material at the sides of the proposed channel rather than to deliver it at a distance through long discharge pipes.

As has already been stated, the volume of material rolled along the bottom is so great as to far outweigh the greatest possible capacity of any dredge. To those who are not familiar with this phenomenon it will seem incredible that a dredge with a capacity of over 1 000 cu. yds. per hour may be operated for days and leave scarcely a trace of the cut. Or, on the other hand, if the cut has been so located as to meet the requirements of the current, a channel will be scoured out and an amount of material removed which would probably exceed the capacity of the dredge many times. It is by no means certain that the channel will always follow the line of the cuts. It sometimes happens that after some days of diligent dredging, a channel develops without assistance on a line quite remote from the dredged cuts.

Whatever is done during one low-water season is generally obliterated by the succeeding high water. The successful location of a cut during one low-water season does not necessarily imply that a similar location during the following season will give like results. In fact, a new problem is presented by each bar every season.

In a stream of this character it is, of course, extremely difficult to determine in a particular case just what share of the channel should be credited to the dredging and what proportion to natural causes. Enough bars have been dredged, however, to fully demonstrate that the dredging at least induces the scour which eventually results in a wide and deep channel.

During the low-water season of 1896 there were eight bars between Cairo and Memphis which obstructed navigation. Channels were successfully opened through all of them except one, which was abandoned on account of an extraordinary change in the river which shifted the channel from the right to the left bank. One of the most interesting of these bars is shown in Figs. 13, 14, 15 and 16. Fig. 13 shows that one day before dredging there was barely a 7-ft. channel between the upper and lower pools. The broken lines enclose the area where the dredging was done. The controlling depth in this area was 6 ft.

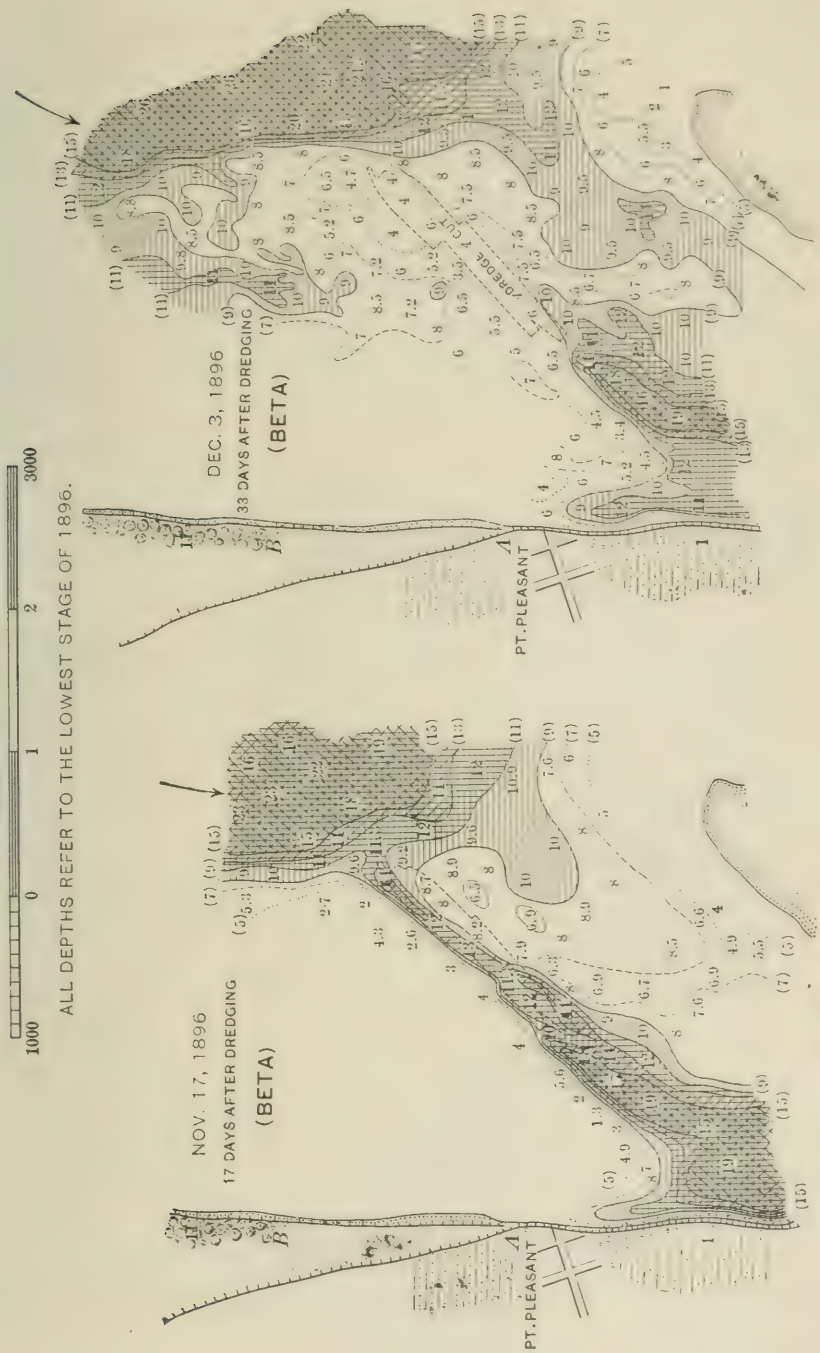

LOWER POINT PLEASANT BAR
79.5 MILES BELOW CAIRO
SCALE OF FEET
1000 0 1 2 3000
ALL DEPTHS REFER TO THE LOWEST STAGE OF 1896.
NOV. 17, 1896
17 DAYS AFTER DREDGING
(BETA)
PT. PLEASANT
DEC. 3, 1896
33 DAYS AFTER DREDGING
(BETA)
PT. PLEASANT

FIG. 15.

FIG. 16.

Fig. 14, two days after dredging was completed, shows the two pools connected by a good 9-ft. channel with 11 ft. of water for the greater part of the length. Fig. 15 shows the conditions seventeen days after dredging. The cut has deepened and an 11-ft. channel from pool to pool, and 13 ft. nearly all of the way through, is found. Fig. 16 shows, in a forcible way, the magnitude of the natural movement of sand bars. Thirty-three days after dredging, the channel was found to be about 500 ft. below the cut opened by the dredge. The site of the dredged channel, which in Fig. 15, only sixteen days earlier, showed 13 ft. of water, now has barely 4 ft. The 9-ft. channel, however, still remains, although it has drifted down stream. This change occurred during a rapid rise at the end of the low-water season of 1896.

A summary of the dredging operations during the season of 1896 is given in Table No. 10.

The following table shows the cost of operating the two dredges during the low-water season of 1896:

Labor	$16 237.06
Subsistence	3 386.67
Fuel	8 051.73
Oil	242.39
Miscellaneous engineers' supplies	176.43
Mates' supplies	94.73
Repairs	855.41
Lighting supplies	114.58
Miscellaneous	113.94
Total	$29 272.94

This only covers the cost of operation at the several bars, and does not include the cost of repairs prior to the beginning of work, the expenses incurred while waiting developments in the stage of river to see whether further work was necessary or not, nor the cost of moving to winter quarters. These items amount to $12 953.23, giving a total for the season of $42 226.17.

During the low-water season of 1897 four dredges were at work on the bars below Cairo. The cost of operating them is given in the table on page 306.

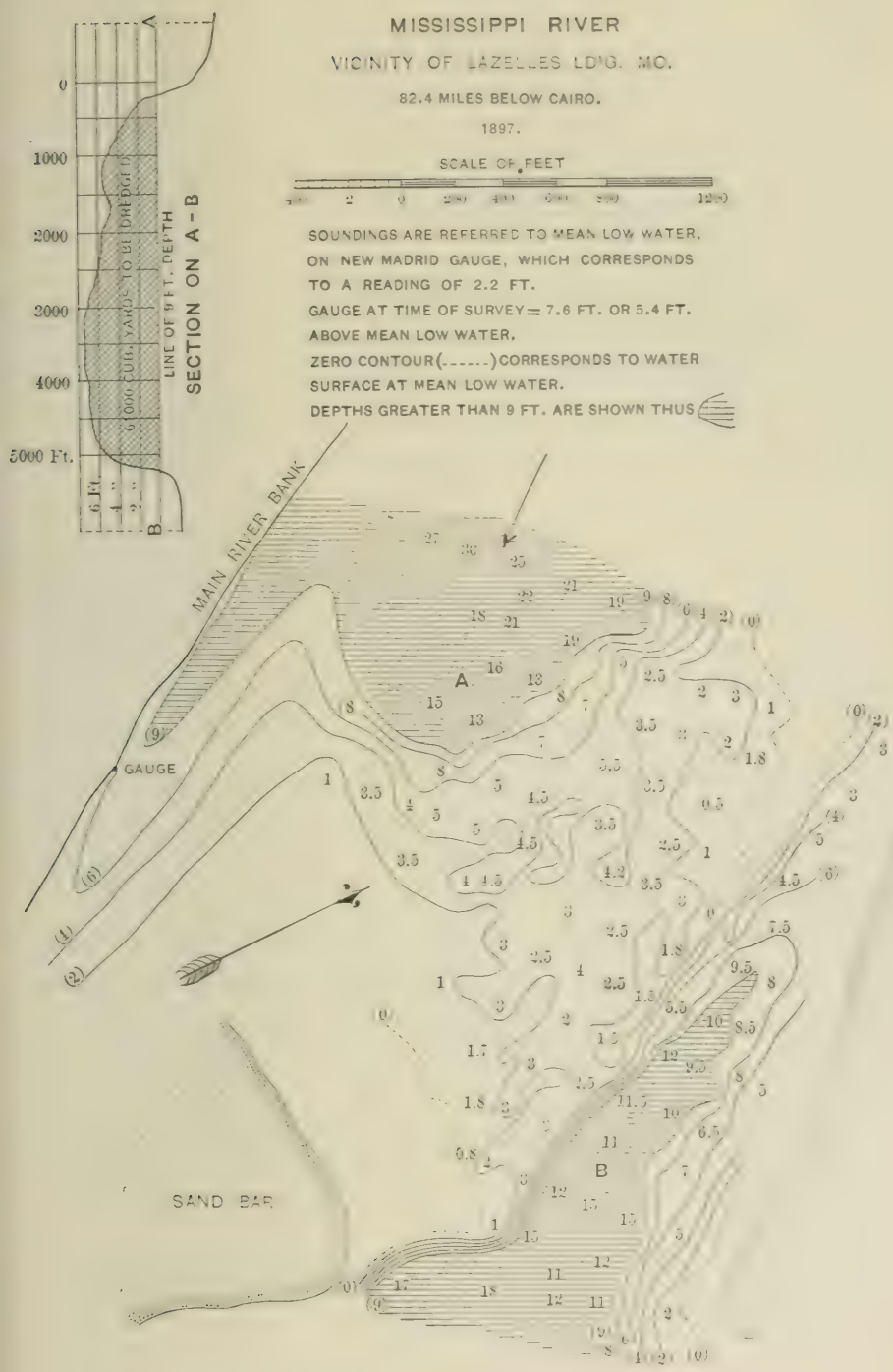

MISSISSIPPI RIVER
VICINITY OF LAZELLES LD'G. MO.
82.4 MILES BELOW CAIRO.
1897.
SCALE OF FEET
SOUNDINGS ARE REFERRED TO MEAN LOW WATER.
ON NEW MADRID GAUGE, WHICH CORRESPONDS
TO A READING OF 2.2 FT.
GAUGE AT TIME OF SURVEY = 7.6 FT. OR 5.4 FT.
ABOVE MEAN LOW WATER.
ZERO CONTOUR(------)CORRESPONDS TO WATER
SURFACE AT MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
SECTION ON A-B
LINE OF 9 FT. DEPTH
6000 CUB. YARDS TO BE DREDGED
0
1000
2000
3000
4000
5000 Ft.
A
B
MAIN RIVER BANK
GAUGE
SAND BAR

FIG. 17.

Cost of Operating Dredges During Low-Water Season of 1897.

Labor	$26 704.53
Fuel	13 474.23
Subsistence	7 586.99
Lubricants	970.79
Lighting supplies	215.05
Miscellaneous supplies	1 527.79
Office supplies	145.10
Hire of tow boats for moving plant	10 115.50
Preparation and moving into field	3 269.44
Moving to winter quarters at end of season	4 637.03
Cost of inspection and supply boat	10 554.75
Cost of operating two survey boats	8 674.10
Total	$87 875.30

Fig. 17 shows a conspicuous sand bar at Lazelles, some 82 miles below Cairo. When the survey of this bar was made, just before dredging, the distance from the 9-ft. depth in the upper pool to the same depth in the lower pool was 1 400 ft. At that time the amount of excavation required to make a 9-ft. channel 250 ft. wide at the datum stage was 61 000 cu. yds. The dredge *Gamma* was at work on this bar ten days, and the actual dredging time was one hundred and seventy-eight hours. Taking the capacity of the dredge as 800 cu. yds. per hour gives the total amount dredged as 142 400 cu. yds. Thirty-one days after the dredging was completed there was a 10-ft. channel clear through the reef, except for a distance of about 225 ft. where the depth was only 8 ft. (see Fig. 18). Seventy-six days after the dredging was completed the channel was found to be 350 ft. wide at the narrowest point, over 1 000 ft. wide for more than nine-tenths of the length, and with a least depth of 9 ft., and an average depth of 11 ft. (see Fig. 19).

At Hathaways, about 104 miles below Cairo, the surveys showed a very ragged 7-ft. channel before dredging. One and a half days after dredging, a narrow 9-ft. channel connected the upper and lower pools, and a clear 8-ft. channel 350 ft. wide extended across the bar. Thirty days later the channel was found to be 12 ft. deep and 800 ft. wide with the exception of a narrow shoal at the lower end.

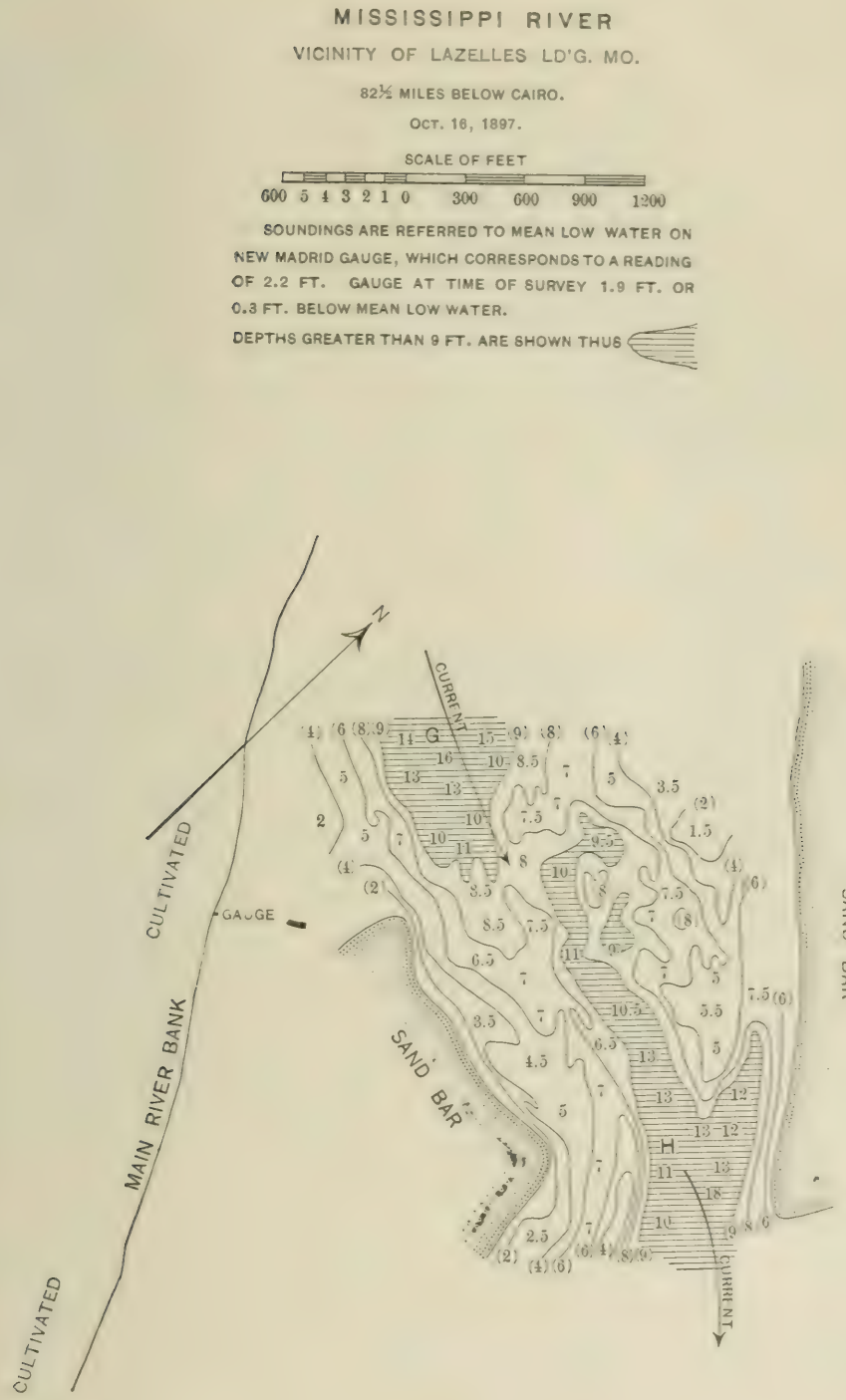

MISSISSIPPI RIVER
VICINITY OF LAZELLES LD'G. MO.
82½ MILES BELOW CAIRO.
OCT. 16, 1897.
SCALE OF FEET
600 5 4 3 2 1 0 300 600 900 1200
SOUNDINGS ARE REFERRED TO MEAN LOW WATER ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.2 FT. GAUGE AT TIME OF SURVEY 1.9 FT. OR 0.3 FT. BELOW MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
N
CURRENT
CULTIVATED
GAUGE
MAIN RIVER BANK
SAND BAR
SAND BAR
CULTIVATED
CURRENT
G
H

FIG. 18.

At Beef Island, 211 miles below Cairo, the channel was deepened 2 ft. in one hundred and six hours of actual dredging by the dredge *Delta*, in spite of the presence of many sunken logs and snags.

At President's Island, just below Memphis, the dredge *Delta* was employed in opening a channel through a reef which was causing much trouble to navigation. In the bed of the river in this locality there is a considerable quantity of conglomerate rock and lignite to be found. The mechanical agitators of the *Delta* were therefore put to a very severe test. One solid piece $3\frac{1}{2}$ x $4\frac{1}{2}$ ft. was caught in the cutters (see Figs. 1 and 2, Plate XXIV). It had to be broken into small pieces to dispose of it. When work began there was barely a 5-ft. navigable channel across this bar. After fifteen days' work, during which period much time was lost in clearing the cutters of debris, leaving only eighty hours' actual dredging time, a 9-ft. channel 100 ft. wide and 1 300 ft. long was completed. This channel remained intact throughout the season, although a wider channel broke through the bar lower down.

During the low-water season of 1897 channels were dredged through fifteen bars lying between Cairo and Graves Bayou, 248 miles below. A view of a dredge at work, showing the pipe line and the discharge against the baffle plate, is shown in Plate XXXV.

There were bars that obstructed navigation below Graves Bayou, as far down as Arkansas City, 435 miles below Cairo, and one bar even below Natchez, 710 miles below Cairo. The dredging plant, however, was too small to cover more than the 250 miles from Cairo southward. The total number of obstructing bars below Cairo during the low-water season of 1897, where the navigable depth was less than 8 ft., probably did not exceed twenty-five. Estimating each of these bars at 2 000 ft. in length, the result is an aggregate of about $9\frac{1}{2}$ miles of bad navigation in a total length of 1 060 miles of river from Cairo to the Gulf of Mexico, or less than 1% of the length.

Estimating the average depth of cut required as 4 ft. and the width at 250 ft., gives the total amount of material to move as 1 852 000 cu. yds. To remove this amount in ten days would require eight dredges running the full time without interruption. Table No. 8 shows that only about half of the time is utilized in actual dredging.

These bars do not all develop at the same time, and they do not necessarily develop at the same places during successive seasons.

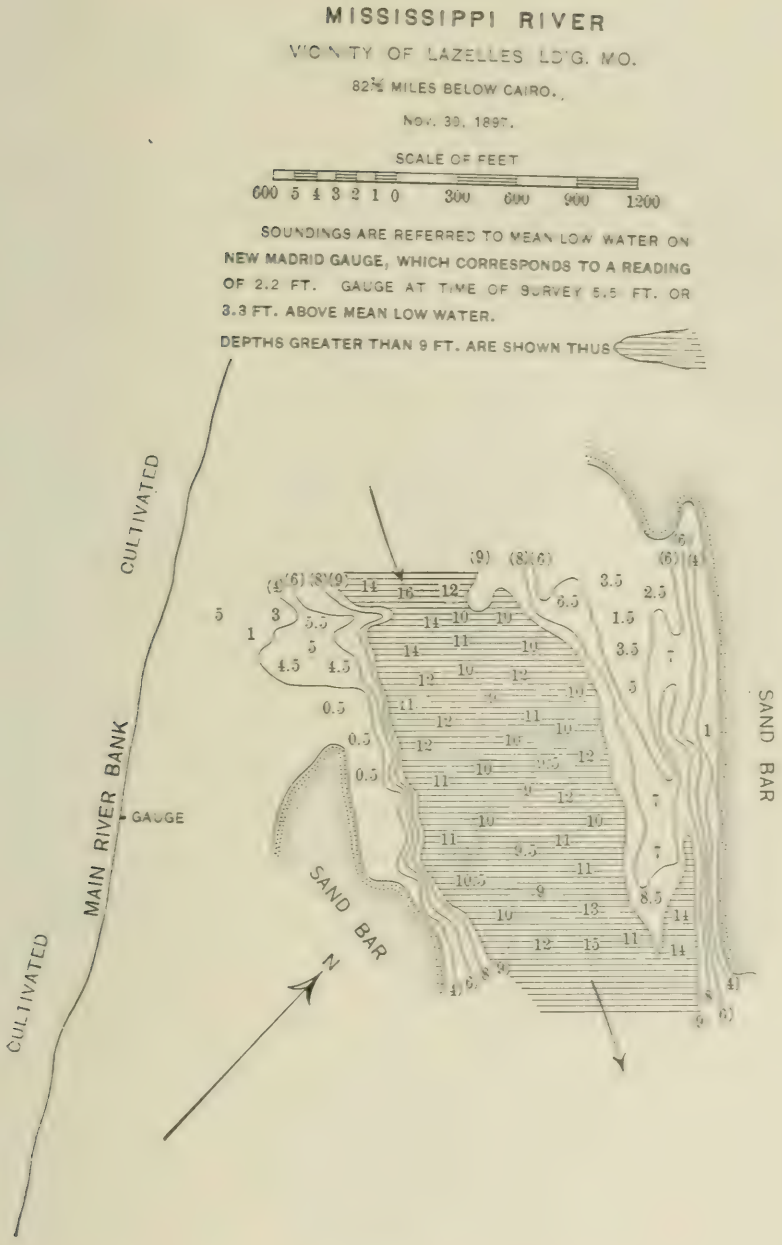

MISSISSIPPI RIVER
VICINITY OF LAZELLES LD'G. MO.
82½ MILES BELOW CAIRO.
NOV. 30, 1897.
SCALE OF FEET
600 5 4 3 2 1 0 300 600 900 1200
SOUNDINGS ARE REFERRED TO MEAN LOW WATER ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.2 FT. GAUGE AT TIME OF SURVEY 5.5 FT. OR 3.3 FT. ABOVE MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
CULTIVATED
MAIN RIVER BANK
GAUGE
CULTIVATED
SAND BAR
SAND BAR
N

FIG. 19.

There are, however, certain general localities that have long been recognized by steamboatmen as liable to develop bad navigation during any low-water season. Now and then, even these places pass through a season without developing any serious obstructions, and at such times dredging below Cairo will be unnecessary. Seasons having good navigation throughout are, however, not frequent.

Dredging operations will, in the main, be confined to the same general localities from year to year, although wrecks, snags and bank erosions may occasionally develop new bars that will require dredging.

The season of 1895 was one of extraordinarily low water. At one time during that season there were thirty-seven crossings between St. Louis and Cairo, where the water was 6 ft. or less in depth. During ordinary seasons about as much dredging would be required between St. Louis and Cairo to secure a 9-ft. channel 250 ft. wide as is required below Cairo for a similar channel. The supply of water above Cairo at extreme low stages is too limited to readily admit of 9-ft. channel depths. The requirements of river traffic on this reach would be satisfied fairly well by 7 or 8 ft., and it is doubtful whether these depths can be materially exceeded by means of dredges or other temporary devices at an expense justified by the value of the results obtained.

It is evident from the foregoing that the problem of keeping navigation open in the Mississippi River during the low-water season is very far from being a simple one. The magnitude of the plant described is sufficient evidence that strenuous efforts are being made to "improve and give safety and ease to the navigation" of the river, and the experience gained thus far justifies the belief that these efforts will be successful.

The writer is indebted to General John M. Wilson, M. Am. Soc. C. E., Chief of Engineers, U. S. A.; to the Mississippi River Commission, to Major Thos. H. Handbury, Captain H. E. Waterman and other officers of the Engineer Corps, for free access to records and drawings. He is also indebted to his associates, engaged in the improvement of the river, for valuable assistance in the preparation of drawings and tabulated data.

DISCUSSION.

B. L. Crosby, M. Am. Soc. C. E.—The speaker has had a great deal of serious trouble with deposits caused by anchoring barges in the current, and although his experience has been on the Missouri rather more than on the Mississippi, yet the lower Missouri may be considered as virtually the same river. Mr. Crosby.

The author states that it is a well-known fact that a barge anchored in shallow water will very soon cause the sediment to deposit, but does not say that it will be formed under the barge—it is always formed immediately down stream from it. In the speaker's opinion, that would be the result with any deflector—that the point of trouble would be simply transferred to a point a little further down stream.

When a long barge is anchored in shallow water, the material scoured out at the bow is sometimes deposited at its stern, so that the barge is aground at the stern and in deep water at the bow.

Edward Flad, M. Am. Soc. C. E.—The speaker agrees entirely with the conclusion reached by Major Harrod, that the water-jet agitator is a more efficient and a better machine than the mechanical agitator, for the conditions under which the dredges *Epsilon* and *Zeta* are ordinarily required to operate. Referring to the tests of these dredges, the author states that it was demonstrated therein that the capacity of the pump with the mechanical agitator was not much over half the capacity of the pump with the jet agitator. The speaker disputes that statement. He was present during all the tests of these two dredges, and acted as engineer for the contractor during the period of the detail designing and testing of both dredges. At first sight, judging from the report of the tests, the author's conclusions might seem warranted, but, taking into consideration the conditions under which these tests were made, his conclusions are entirely wrong. The two dredges were turned over to the Government at the same time to be tested, and the *Epsilon*, which was provided with the water-jet agitator, was tested first; the result being 757 cu. yds. of sand per hour. The contract requirement was 1 000 cu. yds. of sand per hour, and the contractor was much disappointed at the result of the first test, because he had anticipated that the dredge would pump 2 000 or 3 000, or, perhaps, 4 000 cu. yds. per hour. A second test was made and showed 714 cu. yds. of sand per hour, which was likewise unsatisfactory. Mr. Flad.

The tests were made by attaching to the end of the dredge a barge, into which the material was pumped; then the water was withdrawn and the sand was measured. It was observed that the sand which was being pumped was very fine and contained a large number of mud lumps. It was therefore concluded that this was the cause of the reduced capacity, and, as the contract provided that the test should

Mr. Flad. be made under average conditions and with average river sand, it was very properly decided that the dredge should be moved to some other location. Another part of the district, near New Madrid, was chosen, where a very good average river sand was obtained.

All these tests were made under conditions which were calculated to develop the highest capacity of the dredges. Everything was run at the highest speed. The dredges were pulled ahead on wire cables with as great a speed as possible.

The first test in the new location showed 1 712 cu. yds. per hour, and the next 2 800 cu. yds.; the next 3 100 cu. yds. Five tests in all were made on this good material. In determining the average capacity of this dredge the first two tests were, of course, properly omitted, because the material was not what the contract called for. The average of the last five tests was 2 500 cu. yds. per hour, which was very satisfactory.

After these tests on the *Epsilon* were finished, those on the *Zeta* were commenced. During this time the water had been rising, and it was necessary to move the *Zeta* into what is called a high-water sand bar, which is largely filled with drift, logs and lignite, and is a very hard material for any dredge to handle—and especially severe for a dredge provided with a mechanical agitator. However, as the contractor was anxious to complete his test and get his money, he agreed as to the location—the only one available at that time. The first test showed 1 878 cu. yds. per hour, which was thought to be a fairly good showing, the contract requirement being 1 000 cu. yds. The next test showed only 1 300 cu. yds. It was soon found that the dredge was working in a bed of brush and logs, and there was a great deal of difficulty in going ahead. Forcing the dredge to any excessive or even reasonable speed, resulted in breaking the shearing pin of the agitator. However, these tests were sufficiently satisfactory to the contractor, because they were in excess of his contract requirements, and he raised no serious objection to them. The average result of the five tests was 1 364 cu. yds. per hour.

In order to enable one to compare the relative value of the two machines the tests should have been made under the same or similar conditions. Now, as the tests of the *Epsilon*, with the water-jet, were made under very favorable conditions for a high-capacity test, and those of the *Zeta* were made under perhaps the most unfavorable conditions, it was entirely improper to compare the relative capacity on those tests. The speaker believes that the author does not realize the conditions under which the tests were made, because he was not there at the time.

He further states, in reference to the mechanical agitator, "It will therefore be taken out, and jets will be put in." He is mistaken in that statement. The government officer in immediate charge decided that

he would like to remove the mechanical agitator, and his views were presented to the local members of the Commission at St. Louis. They considered the question, and came to the conclusion that, in place of removing the mechanical agitator, they would install, in addition thereto, the water-jet agitator, which was the plan originally contemplated in the construction of the dredges. In fact, the mechanical agitator was so designed that the water-jet agitator could be used in conjunction with it or as a substitute for it, and at the time the dredges were being constructed it was urged upon the Commission that they install the jet agitator in addition to the mechanical agitator. Mr. Flad.

CORRESPONDENCE.

Mr. Harrod. B. M. Harrod, Past-President, Am. Soc. C. E.—This paper is of much interest as a full and accurate description of a new and important development of engineering work.

The method of dredging now in use on the alluvial parts of the Mississippi River has arisen from the inadequacy, when applied there, of methods which have been more or less successful on streams of different character and of smaller dimensions.

The great specific gravity of the material of the bars, which is mostly a clean silica sand, and the distance it would have to travel in suspension before reaching a pool in which it could settle down harmlessly, make any method which proposes merely stirring up the sand, and trusting to the current to remove it, quite ineffective.

The successful results which have been obtained with spurs and dikes in improving the navigation of smaller streams are both doubtful and costly on a river with the characteristics and dimensions of the Mississippi. The bars naturally form in the wider reaches, at the node, or reversion point, between two bends. The banks at such places are frequently a mile or more apart, and the bar, with a 5-ft. crest, has a width, between the contours, of satisfactory depth for navigation (8 to 10 ft.), which must be connected through the intervening shoaler water of a third or half a mile.

While the bars reappear annually in about the same neighborhoods, the changes of location are sufficient to render resurveys necessary each year at the approach of low water; and since the bars and the natural channels through them—generally the latter—may shift up or down during the season, these surveys must be continued until the autumnal rise brings relief.

The same bars, or groups of bars, do not, from year to year, present the same relative degree of obstruction to navigation. A bar which in one year limits the navigable depth of the river to 5 ft., may, in the next, have 8 or 10 ft. over it, and the trouble be transferred to a neighboring locality which, in the previous year, presented few or no obstructions.

The drop during the falling stages is apt to be rapid, and in a week's time the entire barge and through commerce of the river may be arrested. The improvement of the bars should, therefore, begin within the least possible time after the river has reached a stage that will allow the making of surveys necessary for the location of the work, and the work should be of a character that will admit of very rapid application, extension and results.

If it is attempted to meet these difficult conditions with spurs or dike work, it is quite evident that preparation must be made, at each

bar, for building several dikes, each of which may require to be of great length; that similar preparations must be made at many bars; that the character of these works must be temporary or portable; and that such preparations involve the providing of an immense plant of tow-boats, barges and pile-drivers, and a very large force, in order to supply material and erect extensive structures, at many places, and within a very short time. All this is absolutely necessary for a satisfactory low-water improvement of the Mississippi by spur or dike work; and even then, from the shifting character of the bed and banks of the Great River, it does not promise to be more permanent than dredging. Mr. Harrod.

On the other hand, it is quite practicable, and the writer believes it will be found less expensive and more reliable, to have a fleet of such a number of dredges that one could be in waiting, during the falling stages of the river, at each bar where there might be reason to apprehend trouble, ready to go to work, if necessary, whenever a depth of not more than 12 ft. or 15 ft. was found.

During the progress of this work several interesting problems have arisen. One concerns the relative efficiency of the cutter and jet, for disintegrating and delivering the material to the suctions. Of course, in any tenacious material, such as clay, the cutter is indispensable; but it is clearly established that in the loose sand which almost exclusively forms the bars of the Mississippi, the jet is equally effective, delivering as large a percentage of solid material to the pump, while it costs less, weighs less, and requires less power to operate. It is also less liable to accident than the cutter; for when other material than sand is found it is either an embedded drift log, a boulder of concrete gravel, or some other solid object, which no machinery can deal with. From around and under these the jet removes the sand, allowing them to drop below the bottom of the cut.

An additional dredge is now under construction which will be provided with the means of self propulsion, by which addition it is hoped to largely dispense with the services of a tender in moving the dredge from bar to bar. Although the arrangements for moving the dredge while at work on a bar are already very efficient, it is hoped that this operation may also be facilitated.

Experiments are soon to be made with a side delivery pipe, from 100 to 200 ft. long, instead of from the stern through 1 000 ft. of floating pipe which is generally necessary to reach the lower pool. The cut made through the bar is not usually in the exact thread of the current, and there is, therefore, generally an up-stream and down-stream side to the artificial channel. It appears that if the excavated material could be delivered on the down-stream side, it would either be taken away from the cut by the current, or, if it remained where deposited, would act as a training dike, and serve to increase the natural scour

Mr. Harrod. in the channel. The saving would be in the increased output, with the less friction in the shorter pipe.

In ordinary cases the immediate result of opening a channel through a bar, either by dikes or dredges, has been to draw down the upper pool, and increase the obstruction of the bar above; but owing to the dimensions of the Mississippi, the great low-water discharge, the slight fall, the distance between bars, the capacity of the intervening pools, and the small relative increase of channel section required for improvement, there seems to be no reason for anticipating such results. If realized, there is no reason why they should be more intense with dredge than with dike work.

It is true of this work, as it is of all engineering work, that the degree of success is quite dependent upon the judgment with which the artificial channels are located. It is certain that the highest degree of skill, knowledge and judgment is required in the engineer to whom this part of the work is entrusted. To what he has acquired in the schools must be added a close knowledge of the physics of the Mississippi River, which can only be acquired by experience, and also the trained eye and judgment of the pilot. He must detect any natural preference of route across the bar which the river may have; he must know how this can be developed, or modified, to meet the demands of navigation, and must anticipate what movements or changes may occur, either from natural causes or as the result of dredging, to threaten or favor his project.

When this method of low-water improvement was first adopted, the study of the subject was almost exclusively intrusted to the late Henry Flad, Past President, Am. Soc. C. E., who brought to it his large experience, great ingenuity and intense power of application. The first boat was built very largely on his plans. It contained several experimental features, and after these were subjected to a process of selection and elimination, there remained the solid parts which, practically unchanged, have entered into all the later constructions, as described by the author. To Colonel Flad, perhaps, more than to any one else, is due the success of hydraulic dredging as applied to the sand bars of the Mississippi River.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—This excellent paper is a very welcome contribution to the knowledge of river works, and announces good news in the way of very important steps taken in the direction of speed and economy in accomplishing the great problem of maintaining good low-water navigation, at a moderate annual expense.

The engineering troubles mentioned are on a stupendous scale, as compared with those encountered on ordinary rivers, but are, nevertheless, quite similar to those experienced on all rivers having a large amount of sandy bed flow. Each year, as the flood waters subside, and the low-water stage approaches, the wide sheets of shallow water,

passing over the crests of the reefs, begin to subdivide into channels, more or less marked, the locus naturally selected being at places where the material on the reef happens to be more easily eroded than elsewhere. Should several channels develop across the reef, with no particular difference as to depth or width, the judgment of the engineer is often sorely taxed to tell which one of the number to select for improvement. On one hand, free navigation demands a reasonably straight crossing, while on the other, experience shows, and it too often happens, that the square crossing at right angles to the general trend of the river, is nevertheless the easiest and most reliable channel to improve. This abnormal position arises from the natural fact that, as a rule, the lightest material is generally, if not always, deposited at the up-stream flank of the reefs. All an engineer can do, in such tentative works, is to study and cautiously follow in the footsteps of natural erosion as the waters subside. To do otherwise would be courting humiliation and wasting money. Mr. Le Conte.

The Long scraper mentioned by the author has been used with success on the upper rivers in California in deepening chutes across gravel bars; but in the lower rivers, having wide sandy crossings, the use of the scraper has not been a success.

The temporary wing dams mentioned by the author are certainly very important inventions, which will undoubtedly be of great value in the general conservancy of low-water navigation. They are capable of doing a vast amount of work in a short space of time, the main points to be considered. The practical application must be tempered with much judgment, however, otherwise new bars will be developed by the disturbance of the level of the pool waters, and the new bars created may require a large amount of unexpected work.

Tree-dams, referred to in the paper, constitute a very cheap and effective means of improvement; the up-stream ends or butt-ends being anchored down on the line of the dam. Such dams, of course, last for the season only, as a rule, and are washed out by the following flood waters; but in some cases it is practicable to build them on the making side of the bar, and thus the useful life of the dam may be prolonged many years by making small annual extensions.

The writer is particularly interested in these important dredging results, inasmuch as he has been closely associated with hydraulic dredging plants at the Oakland harbor-works, California, since 1883. His first paper on this subject was read before the Society on December 5th, 1883.* Since that date every imaginable variety of "cutter" has been tried with more or less success in its particular line of duty. The hydraulic jet admits of wide application, and is eminently suited to bring about the complete disintegration of all classes of material, except rock and tough rubber-like blue clay.

* *Transactions*, Am. Soc. C. E., Vol. xiii, p. 9.

Mr. LeConte. There is a great variety in the style of centrifugal pumps in use, and this variation in practice is due chiefly to lack of comprehension on the part of those who may be called clean-water pump men in contradistinction to the dredge-pump men. The former always insist on the use of small runners at high speed, whereas experience shows, that for dredging work large runners, with faces as wide as the diameter of the suction pipe, and moderate speed give the best results with lowest costs for wear and tear. As soon as the water column becomes heavily charged with material, sand particularly, the element of friction is developed enormously, which fact weighs heavily against the use of small-sized pumps. Again, dredged material is notoriously full of all kinds of foreign matter, and, as a rule, all that which enters the suction pipe must pass through the pump. Any attempt to intercept the same by straining devices will surely end in failure due to clogging. The exception is in the case of stone, which can be caught in a depressed trap. The reverse-curved arms have been used in the Australian dredges built by George Higgins, M. Inst. C. E., and are highly spoken of. The runners of the larger ones are 8 ft. in diameter, and have developed unusually high efficiency at moderate speed, a matter of great moment for dredging.

Mr. Wheeler. W. H. Wheeler, Esq.—Reference has been made in the paper to the removal of shoals in rivers by a mechanical system of erosion, and the transport of the material eroded by the action of the water; a description of some of the devices which have been used from time to time for this purpose being given. The author's conclusion appears to be that none of these machines has satisfactorily answered the purpose for which it was intended.

Having spent a great deal of time in experimenting and investigating the best means of applying the transporting power of water to the deepening and improvement of river channels, the writer is pleased to have the opportunity of stating the results at which he has arrived.

It is a matter of common observation that all rivers during land floods are charged with a large quantity of alluvial matter which is carried to the sea in suspension; and their turbid condition then is practical evidence of the ability of water to transport material. At other seasons, when the water flows away to the sea comparatively clear, there is a waste of energy when there are shoals in the bed of a river which require to be removed, or where the channel can be improved by being deepened.

Flowing water has generally a large amount of undeveloped power, and glides over shoals without exercising the erosive action due to the velocity at which it is running. A very slight cause will lead to the disturbance of the particles of which such shoals are composed, and will place them in suspension in the water. This latent power may, therefore, be changed into active energy by mechanical erosion and

diffusion, and thus the river may be made the agent for its own improvement. Mr. Wheeler.

It is unnecessary to enlarge on the capability of flowing water to transport material, as this subject is fully dealt with in the chapter on the transporting power of water, in "Tidal Rivers."*

An investigation of the various machines which had been invented for the purpose of mechanical erosion led the writer to the following conclusions: These machines failed either, (1) in completely churning up the material and incorporating it with the water; (2) in reducing the particles to a sufficient degree of fineness to allow them to remain in suspension after being once set in motion; or (3) by creating a false current, in the vicinity of the machine, which carried away larger particles than the normal velocity of the stream could hold in suspension, and these particles, therefore, settled in the channel as soon as they got beyond the sphere of the increased current. Harrows and similar devices fail to fulfill the first and second conditions, and screws of steam boats, which, under certain conditions have been found of use in disturbing and breaking up shoals, come under the third condition.

The type of eroder evolved, after a series of experiments with models of various shapes and designs, consists of a cone-shaped cutter, having a number of steel blades fastened vertically on the periphery. This cutter is attached to the end of a vertical shaft connected by bevel gearing with another shaft carried on a horizontal ladder working in a well in the center of a barge, and driven by a steam engine on board. The ladder, which is hinged at the upper end, rises and falls as required in order to adjust the cutter to the depth of water in which it is working. The barge is warped forward and sideways by chains running on winches worked by the engine. The cutter works against a fall of from 2 to 3 ft., and, as material is detached by it from the shoal, the pieces are rapidly rolled round and round by the centrifugal current created by the cutter, and rubbed against the face of the shoal until the particles are reduced to a size of sufficient fineness to be carried away by the current. By this means the cutter automatically adjusts the size of the particles to the velocity of the stream, the particles not flowing away until they are capable of remaining in suspension in water having a less velocity than that of the stream into which they are discharged. If the current has a low velocity, say 1 mile an hour, the particles continue to be rolled about in the vortex at the cutter until they are reduced to a size of 1 000 to 2 000 to a lineal inch. The cutting of the face of the material to be removed is more due to the disintegrating effect of the small pieces of soil being continually rubbed against it by the centrifugal current than to actual

* "Tidal Rivers, Their Hydraulics, Improvement and Navigation," by W. H. Wheeler.

Mr. Wheeler. contact with the blades of the cutter. The material thus placed in suspension has been carried to sea for a distance of several miles without any redeposit in the channel.

This eroder gives the best results when working in shoals of alluvial matter or in clay. In sand shoals the particles are too large to remain in suspension in the channel after leaving the eroder. The machine, however, has been successfully used in deepening shoals consisting principally of coarse sand and shells mixed with gravel. In this case, the disturbance caused by the eroder set the finer particles in motion and mixed them with the flowing water, the coarse sand and gravel dropping to the bed of the channel, which, for the purpose, was excavated to a greater depth than that required for navigation.

A machine made on the principle here described, and placed on a steel barge built for the purpose, has been successfully at work for the last eight years in removing shoals of alluvial matter which make up, from time to time, in certain parts of a tidal navigable river which the writer has under his charge. Owing to the position of these shoals and the difficulty in removing the material, the cost used to amount to about 18 pence per cubic yard. With the eroder in use, the cost has been reduced to 1 penny per cubic yard. In other parts of the same river shoals consisting of compact beds of clay, mixed in some places with chalk, and in others with peat, have been eroded and cleared away to depths varying from 1 to 5 ft., at a cost of from 1½ pence to 2 pence per cubic yard measured *in situ*.

The eroder commences working on these shoals on the ebb tide, as soon as the water has fallen sufficiently to allow the ladder to reach the bottom.

It is not contended that such a machine is adapted to rivers of the magnitude of the Mississippi, or in situations where the amount of work to be done warrants an outlay on a large plant. In such cases, suction dredges, fitted with cutters and delivering into hoppers, to a certain extent fulfill the same purpose as an eroder, as the smaller particles of material disturbed by the cutter either float away directly in the stream or in the effluent water from the hoppers. For this reason, when there is sufficient current and the river water is not in a turbid condition, the more the cutters disintegrate the material, the greater will be the proportion of it transported by the stream, and the less the proportion to be carried by the hoppers. It is due to this cause that, in many dredging operations in silt and alluvial matter, it is found that the quantity carried away in the barges is less than that measured *in situ*.

There are, however, numerous smaller rivers, where the traffic is not sufficient to warrant the outlay required for the purchase of expensive plant, nor the revenue sufficient to pay the cost of hiring and working ordinary dredges, and in these the navigation could be materially improved by an eroder of the kind here described.

W. S. MITCHELL, Esq.*—In the effort to deepen the low-water channels of rivers flowing over shifting bars or "crossings," it is probable that temporary or portable dams or jetties were suggested at an early date. Their practical use, however, is not believed to have been extensively tried in a large river until, within the past two years, in the Mississippi between the mouths of the Missouri and Ohio Rivers, where they were used on such a scale as to warrant a fuller description of their evolution and effects than is given by the author in his notice of them as "current deflectors," which, by the way, they are, only in a limited sense, their "deflecting" effect being of minor importance. They have been used as "contractors" to cut off a portion of the cross-section by giving an artificial crest to the bar on which they are placed, as close to the steamer channel as possible; this slight damming resulting in an increase of a few inches in the slope over the shoal, and this has been sufficient to gradually deepen the channel across it. Mr. Mitchell.

It is true that in placing them, the jetties have usually been given an inclination down stream toward the end of the shoal, but, although various angles have been tried, this has proved of little value. Latterly, the angle has been made very flat, and no hesitation has been felt in changing the direction of the jetty, or any portion of it, to take advantage of the irregularities of the river bottom and thus avoid crossing very deep water. A depth of about 10 ft., with good current, has been found to be the practical limit in which such light and temporary structures can be placed quickly and cheaply. They have been successfully carried across depths of 16 ft. to 20 ft., but the piling supports and foot-mats have had to be so greatly increased in strength that the cost was excessive.

These structures, in their evolution from plank aprons hung over the sides of barges or small flat-boats moored in the desired lines, to corrugated steel panels resting on the bottom on small fascine-mats, or plank-mats and supported at their tops by piling connected by stringers, have been fully described in the reports of the Chief of Engineers, United States Army, for 1895, 1896 and 1897, and will receive further notice in that for 1898.

In all, on fifteen bars, twenty jetties, aggregating more than 3½ miles in length, have been placed. In the past two years, 10 000 lin. ft. of steel panels (costing about $3.50 per lineal foot or 20 cents per square foot) have been kept on hand for use as needed.

In the low-water season of 1896, 6 500 lin. ft. of jetties were built in seven lines on six shoals, and in the fall of 1897, 9 300 lin. ft. were built in nine lines on seven shoals; their cost having been reduced to about $3 per lineal ft. of jetty constructed. This includes all charges for superintendence, labor and plant, placing, maintaining, removing

* United States Assistant Engineer, St. Louis, Mo.

Mr. Mitchell. at close of season, storing when not in use, repairing, loss and deterioration of panels and plant, towage, etc.

A party of seventy men with three pile-drivers and two derricks will place about 250 ft. of jetty per day of eight hours, and in the same time will remove about 400 ft., including the drawing of all piles and loading both piles and panels on barges.

In placing the jetties a great variety of positions has been tried. A few have started from the river bank or from dry bars, jutting out into the stream, but most of them have been placed in mid-stream as close to the channel as possible, and usually, though not always, on the lower side of it, leaving the latter to pass at either end it might choose. At Danby Landing a jetty 1 200 ft. long was placed, with the channel passing in the 300-ft. space between it and the Missouri shore, and the result was the silting up of that passage and the formation of a new channel close to the jetty in the 700-ft. space east of the line. At Platin Rock the jetty (2 200 ft., the longest yet built) preserved and deepened a channel that passed directly under its full length. Jetties have also been tried in series like spur dikes, and in pairs acting as funnels.

In general, their effect has been remarkably good, the channel depths being increased by from 1 to 3 ft., and also being maintained, even with a steadily falling river, which in the past year reached the extraordinary stage of 1 ft. below standard low water.

In addition to the deepening of the channel proper there is always considerable scour immediately above, but so close to the structure that it cannot be used as a channel even by the smallest boats, as it is rarely more than 30 ft. in width. It is this cut that is usually dilated upon by those river men who are advocates of various forms of jetty identical in principle, such as lines of sunken barges, barges from which heavy sheet piles are dropped, etc., but it cannot be used while the jetty is in position; and, on the removal of the latter, it immediately silts up.

Another claim by these advocates is that their forms of jetty will accomplish the desired work in a few hours or days, and that the plant may then be removed for use at other places, leaving a permanent and constantly deepening channel through the bar last occupied. Unfortunately, experience does not confirm this, but, instead, it has been invariably necessary to keep the jetties in place during the entire season to maintain the gain in depth secured by their use, and they have been removed only on the approach of ice. To do this with the forms referred to, and on the scale needed, would involve such enormous outlay for plant and operation as to be prohibitory.

Jetties worked in conjunction with some of the several forms of dredges have been tried and are very successful in maintaining the depths secured so much more quickly in that way. This has sug-

gested the thought that it may be possible to build a jetty, either in whole or in part, of the sand thrown out by the powerful suction dredges now coming into use. If this can be successfully accomplished, it will greatly cheapen and simplify the maintenance of dredged channels in such rivers, because, as a result of the moving sand of the river bottom, their tendency is to fill up at once upon the removal of the dredge. Occasionally a dredge-cut, without other work to maintain it, may deepen and improve, and become the permanent channel for that low-water season, but more often it will require a repetition of the work of the dredge, which will thus be forced to patrol its district, cutting channels wherever needed and repeating the work as many times as may be necessary. Mr. Mitchell.

In addition to the jet dredge noted by the author (pages 238, 239), good results have been secured by the use of the wheels of powerful tow-boats anchored on the bars by spuds. The wheel is turned at any desired rate (usually $\frac{1}{2}$ to $\frac{3}{4}$ full speed), the race of water washing the sand rapidly down stream. Depths of 8 ft. to 10 ft. have been quickly attained in this manner. The tow-boat *General Abbott* is fitted with two powerful duplex pumps, 20-in. and 12 x 24-in., discharging through the stern rake just forward of the wheel, through two 4-in. stirring nozzles lowered close to the sand, to aid the wheel in cutting into the bottom.

This boat has been very useful in cutting off the points of reefs and in removing mud-lumps, thus straightening channels which otherwise were tortuous and difficult.

All these jet, stirring or washing dredges, however, are at distinct disadvantage on wide bars, in having to move material more than once before its final disposition by the current in the nearest pool below the shoal.

The two new suction dredges mentioned as designed by Major Handbury, for use between the Missouri and Ohio Rivers are now ready for service. They are powerful dredgers, and that they may be expected to do effective work during future low-water seasons, is justified by their highly successful inspection trials when, under working conditions, a maximum discharge of 1 200 cu. yds. of solids per hour was obtained with each.

The swivel joints in the discharge pipes at the stern of the dredge and the pontoons, arranged to swing under the pipe, are believed to be novel and a distinct advance toward the solution of the problem of handling the long discharge lines of suction dredges.

By setting the pontoons at an angle with the current ("lee-boarding" as it is termed by the river men, and they may be so set, singly or all together by means of rope bridles running from the pontoons to the dredge capstans), the whole pipe line has been swung out at an angle of 48° from the dredge, and a spread of 80°, or 670 ft. between

Mr. Mitchell. the ends of the two lines, has been obtained. With greater experience, this distance can probably be increased to 800 ft., thus enabling the discharge to be placed 400 ft. on either side of the dredged channel without the use of piles or lines to haul the pontoons into the desired position.

Mr. Coppée. H. St. L. Coppée, M. Am. Soc. C. E.—The dredge, aside from the simple act of taking from the bottom or sides of a ditch or water-way a certain given amount of material, has had a moral and political influence hardly secondary to the railroad.

In addition to reclaiming harbors, and deepening and regulating rivers and canals, it has hurried the advance of civilization by increasing the facility for navigation and commerce between far distant lands, cutting the very sinews of race prejudice and religious intolerance, and bringing the antipodes together.

Contemplate the East of to-day and prior to 1859, when Ferdinand DeLesseps struck the first blow on that barren neck of land that ten years later was divided by a canal connecting the Mediterranean and the Red Seas—the new and brilliant West with the shadowy Orient. European civilization, customs, dress, thought and education, then but surmised in that land of compulsory ignorance and superstition, are now almost universal.

The history of dredges and dredging, and their evolution from the very crude beginnings in the not remote past to the great machines described in this valuable paper, is extremely interesting.

The *Century Dictionary* gives the root of the word dredge as *drage*, meaning to draw or drag. The first dredging tools were purely drags, drawn along the bottom. To-day dragging plays no part in the operation of dredging with the modern machine.

The principles which govern the operations of the various dredges were probably applied daily by primitive man, and most of the types now in use were illustrated by his actions before the dawn of civilization. He is seen in his endeavor to enlarge the basin of the spring that bubbles from the earth near his rough abode, scraping the sand or leaf mould from the bottom, making of his foot and hand a crude scraper dredge; or perhaps holding with one hand to a tree or vine, and, with the other, scooping up the material and throwing it on the bank, as in the case of the scoop or dipper dredge; or standing astride the pool, reaching down his muscular arms and clasping double handfuls of earth as the clam-shell digger does; or, where the pool is more extensive, and its waters supply a community, with his fellows forming a human conveyor to transport the more tenacious material from its bed to the bank, naked arms, hands and bodies performing the functions of an endless chain.

This latter mode of dredging was practiced as late as 1864 on the

Suez Canal in the incipient stages of the work, when Fellah labor was furnished by the Khedive of Egypt. Mr. Coppée.

Consul Farman states, in an interesting report to the State Department, May 18th, 1881:

"Two short canals, which supply the City of Alexandria and the small District of Beharah, required for their ordinary cleaning each year twenty-five thousand to thirty thousand persons, for a period of from forty to fifty days. One of these canals was re-excavated by Mohammed Ali in 1819 to 1820, who employed in its construction two hundred and fifty thousand (250 000) laborers for about one year, of whom twenty thousand (20 000) are said to have perished from sickness and hunger. All the labor cleaning the canals and repairing the dikes is obtained by what is known as the Corvee, a system of forced labor. The inhabitants of the Fellah class are taken in mass from their villages and compelled to work forty-five days or more, if required, under overseers and without remuneration * * * Women and all the boys and girls down to the age of eight or nine years are seen carrying baskets of earth on their heads."

The canals that are dug are cleaned by taking out the accumulation of deposit in baskets, carried, for the most part, on the heads of the women and children. In many places there is considerable water and a very thick mud deposit; this is removed by a man standing in the water and working out with his hands pieces as large as he can conveniently raise, which he tosses to his neighbor, standing near to or on one side of the bank, whence they are sometimes passed to a third or even a fourth person, until they are removed to a sufficient distance. "Even at the average pay for agricultural labor in Egypt of 10 cents per day, such dredging would not be excessively cheap."

Again, in his report on the Suez Canal*, Professor J. E. Nourse, U. S. N., states:

"The lake to-day is a large marsh of salt water resting on clay; in spite of its character, it has not proved specially unhealthy. The line of the canal here for forty-four kilometers presented difficulties in place of less work, for its varying depth presented obstacles in the way of any connected work; but the vigorous race of fishermen on its borders, accustomed to the sun and the mud, scooped up the clay in their hands, rolled it into balls on their chests, and then carried it off on their backs with their arms crossed behind. In this way they raised about four hundred thousand (400 000) cubic meters."

In some parts of China, where even at the present time can be seen the customs and methods of an ancient world, the dredging and levee building is, to a considerable extent, carried on in the same primitive manner.

The writer is indebted to Captain W. E. Hunt† for the following description of a dredging outfit, in a letter received in January, 1894: "They attached a bamboo pole to a scoop-shaped wicker basket; the men and women have a mode of fastening it to their backs and scoop-

* Senate Ex. Document No. 198. 1884.

† U. S. Ex-Consul to Hong Kong.

Mr. Coppée. ing up the material from the bottom. From the basket they throw the mud out into a small boat, the whole family, children included, assisting in the operation. (The family lives on the boat.) After obtaining about one hundred pounds of mud, they pole the boat to deep water, where it is discharged."

In Mr. Rickard's article, "Gold Dredging in New Zealand,"* he states that there was in use about thirty odd years ago a spoon made of a piece of hide fastened to a ring of iron connected with a shovel blade and attached to a long handle, with which the gold-bearing sand was scooped from the bottom of the river. At points where the river became too deep for wading, a punt was employed, and the hand power was aided by winch and tackle (not unlike the Chinese method).

What a contrast to the mammoth dredges of Europe and America, which in a day, with their huge arms and teeth, plow deep channels through the solid earth, or with the suction of their great centrifugal pumps, draw from their resting place hundreds of tons of the ocean's sands.

Undoubtedly in the ancient civilizations that have sprung up, matured and passed away, leaving but faint traces of their greatness, engineering appliances of many kinds were used, among them the dredge in some form. That this dredge was more than a crude type of the existing machine is hardly possible, for in the very seats of ancient prosperity, commercial and intellectual, there are found to-day the most primitive machines, no trace of former greatness in engineering methods or appliances being discernible. The harbors of the Phœnicians, the first great navigators and commerce carriers, have been so obliterated by the ocean, that to-day it is said, with one exception, there is no safe port for vessels of modern times in any of their formerly great roadsteads.

In a very interesting paper on the construction and use of the hopper dredge, by Alexander Charles Schonberg,† are to be found some valuable notes on the history of dredges. Mr. Schonberg states that over one thousand years before the Christian era, artificial harbors existed that were probably originated and maintained by the Phœnicians, as they were the only great navigators at that early period. Their boats were flat bottomed, drawing about 4 or 5 ft., and of 50 to 60 tons burden. In these small vessels they carried the rich products of the soil, and of art and manufacture, to the remotest parts of Europe, Asia and Africa, founding great commercial ports on many of the great water-courses of the Eastern Hemisphere.

The origin of the harbor on the coast of Great Britain, Mr. Schonberg states, was a crude haven of refuge, constructed by piling up a

* *Engineering Magazine*, June, 1894, p. 349.

† Read before the Civil and Mechanical Engineers' Society, England, March 14th, 1888.

few rocks, or other material, as a breakwater, behind which these small vessels rested during stormy weather. In time, this small wall was pushed out to deeper water, its dimensions being increased until it finally protected a harbor of large extent and great commercial importance. Perhaps, behind these walls in those early days some simple kind of dredge was used, but there is no authentic record of such. After the Phœnicians, the Greeks and Romans took their place as the navigators and commercial carriers of the eastern world, building sea and river works to further their projects of conquest and gain, and on the decline of the Roman power, the Venetians continued the construction and improvement of harbors. Still, there is found no record of dredging operations (though undoubtedly such were carried on). Mr. Copper.

Weale's papers on Engineering (1843) notice a work by Verantuis, bearing date 1591, in which there is described a dredge consisting of two pontoons lashed together with two spoons between them working in opposite directions by means of a walking wheel, the raised material falling into a hopper barge. This is considered the earliest form of the primitive dredging machine.

From the crude tools, paddle, rake, scraper, plow, harrow, etc., used in the sixteenth century in England and on the Continent, little advance was made until the close of the eighteenth century, when steam was beginning to be applied to machinery, dredges included.

In 1796 it is stated that a Mr. Grenishaw, of Sunderland, applied to Messrs. Boulton and Watt for a steam engine to run dredging machinery for the purpose of cleaning Sunderland Harbor. The machine was built in the following manner: A 4 H.-P. engine was placed in a flat-bottomed boat 60 ft. long, 20 ft. wide and 6 ft. deep, with a draft of 4 ft.; the weight of the engine and machinery was 23 tons; the capacity was four spoons, each containing 1 ton of material when full, lifted to the height of about 10 ft. per minute. With the exception of the crude machine mentioned by Schonberg as being invented in 1618, this is probably the first steam dredging machine ever constructed. The spoons in this dredge were in the form of truncated cones, with the small end closed, the open end having a spade bit. The spoons were made of hide leather with an iron rim (somewhat similar to the New Zealand scoop before described). The engines had a $12\frac{1}{8}$-in. cylinder and 3-ft. stroke, and made 30 strokes per minute, and were of the form called bell-crank engines.

The editor of *Engineering News* states that Oliver Evans in 1801 built in America a floating dredging machine operated by the first successful double acting high-pressure engine ever made. By putting wheels under his boat and connecting them with his engine, he propelled this nondescript craft $1\frac{1}{2}$ miles overland to the point of launching.

With the exception of one type of dredge, which seems to be indigenous to America (the dipper or scoop), the history of dredging in

Mr. Coppée. the nineteenth century has been quite similar on both sides of the Atlantic. First the scraper, plow, jet, etc.; then the clam-shell digger (also the dipper in America), then the bucket ladder, followed by the hydraulic dredge; each type having numerous modifications to suit the conditions under which they operate and the special duty for which they are designed.

The first use of the dredge in the United States seems to have been in the construction and maintenance of canals.* Canal work is mentioned as early as 1770. The James River Canal was constructed in 1785 and the Dismal Swamp Canal in 1794, the Merrimac River Canal in 1792 and the Carondelet Canal in Louisiana in 1794, the Charleston Harbor and Santee River Canal in 1802. Until 1824 there was no established United States Engineer Corps, and up to that time little or no work on river or harbor improvement had been undertaken.

William P. Craighill, Past President Am. Soc. C. E.,† states that as late as 1853 in this country there were few dredges. The Government advertised for bids for dredges to improve the harbor of Baltimore for two or three years, but no contractor would submit a reasonable bid. It was, therefore, necessary to construct dredges of the Osgood pattern, also dumping scows and a tugboat. The Osgood dredge was the simplest form of the scoop dredge, the first steam dredging machine generally used in America. Thus it is seen that the more general use of dredges proper and their development to machines of great perfection, to operate under varying conditions, is confined to comparatively recent years. In little more than a century the crude rake of the Clyde contractor has been changed for the most complete modern dredge, moving in some instances as much as 10 000 cu. yds. in twenty-four hours.

A few years ago (1872) the hydraulic dredge was hardly more than the embryo of the machine in its present fully developed form; the simplest application of the principle consisting of a barge, a centrifugal pump with suction and discharge of the simplest form. The dredges of the Mississippi River Commission are probably the largest and most effective in the world. The value of their work is no longer problematical, though, on one or two crossings, they have not been altogether successful. On a great majority they have been entirely so. The trouble seems to be, not in the work of the dredge, but in the difficulty of locating the cut through the crossing to the best advantage, and foretelling the action of the currents, both forming and reducing the bar. All the little idiosyncracies of the great river where these dredges work may not be known, but as time goes by and experience and data accumulate, the general and perhaps specific law of

*"The Beginnings of Engineering," by J. E. Watkins, Assoc. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxiv, p. 309.

† "Some Observations on the Subject of the Improvement of Several of the Rivers of the Atlantic Coast." *Transactions*, Am. Soc. C. E., Vol. xix, p. 234.

flow over the bars will be developed, and less power and time will be wasted in accomplishing greater results. Mr. Coppée.

The author has not given the cost of dredging with these machines, probably because of the difficulty of determining what material is actually excavated and conveyed by the dredge and what removed by the current.*

It is interesting to note that the jet agitators have proved more satisfactory than the mechanical cutters. It was the opinion of the writer that the reverse would be the case, but the reduction in weight and care of machinery, and the effective work of the jets, have proven beyond all doubt the superiority of the latter on the sandbars of the Mississippi. In fact, in a number of instances, the dredges have worked well without either.

When the great dredge *Beta* was completed, tests of its capacity were made, as shown by the author, most of them lasting less than a minute. These tests were very misleading. After the dredge was put to work, its capacity for continuous operation was found to be far under the figures given by the short-time trials. In the case of the new dredges, the capacity and efficiency tests are also of short duration, and are but a poor indication of what the dredges will actually accomplish under the average conditions of work for which they are designed.

It is the writer's experience that very inferior dredges will often give great results for a few minutes under favorable circumstances, but will show weakness and inefficiency in a day's run.

The changes to be made in the *Beta* will add to its effectiveness and should reduce the cost of operation. The great draught, augmented by the weight of the cutter engines and other machinery that will be removed was much to its disadvantage. As the author states, the wear and tear of the cutter machinery, to say nothing of the power required, added largely to the expense.

The writer examined this dredge about the time it was tested, before being accepted from the contractor, and was much impressed by its great size and powerful machinery, and the perfection of detail, which is rarely seen in machinery subject to such shocks.

A. F. Woolley, Jr., Esq.—The dredge *Ram* was built in 1893 to work in what is known as Old River—virtually the mouth of Red River. The conditions under which it was to work were quite different from those which would be encountered in the Mississippi, the classes of material to be handled running through the various grades of very soft or semi-liquid mud, soft mud and sand mixed in strata, stiff blue gummy clay, gravel and fine, medium and coarse sand. Mr. Woolley.

In 1831 the Mississippi River, with artificial assistance, made what is known as "Shreve's Cut-Off," passing across a narrow neck of land

* For summary of operations, including cost, etc., see Chief Engineer's Report, Miss. River Commission, 1897, p. 3 612.

Mr. Woolley. and making an island of the former peninsula, which is now known as Turnbull's Island. Red River formerly emptied into the Mississippi at the upper end of the sharp bend forming the western extremity of this peninsula, and the Atchafalaya River was a small outlet of the Mississippi at the lower end of this bend. The old bed of the Mississippi has been filled in by the annual deposits, until there remains only a narrow and inadequate channel which hugs first one and then the other of the original banks of the old river-bed.

From the original mouth of Red River, around the upper or north side of Turnbull's Island, to the eastern end, is known as Upper Old River. From the original mouth of Red River, around the western end, to the Atchafalaya is called Red River; along the southern side of the island and out to the Mississippi is known as Lower Old River, and through the latter navigation is maintained.

It will readily be seen that at such times as the Red River is at a higher stage than the Mississippi, Lower Old River is carrying a portion of Red River to the Mississippi in the capacity of a tributary of the latter, the Atchafalaya carrying the remainder to the Gulf, *via* a a chain of lakes, and emptying 20 miles south of Morgan City, La. At such times as the Mississippi is at a higher stage than the Red, the Lower Old River becomes an outlet of the Mississippi, carrying a portion of its waters to the Atchafalaya, and in this case the Red and Old Rivers are both tributaries of the Atchafalaya.

It is to be noted, therefore, that sand is carried into Old River to a distance and in quantities varying with the velocity of the current, which is in turn quite erratic and dependent on the fluctuating stages of the Mississippi and Old Rivers, the maximum velocity being 10 miles per hour. The direction of the current is not infrequently reversed for short periods, shoaling up and thereby aggravating the evil from drifting sands, a portion of which would have passed through Old River and a larger portion of which would have been distributed over a large area, or deposited in some of the deep pools to be found in Old River.

There are two other and more serious obstacles to maintaining navigation, viz., the large sand bar annually formed at the junction with the Mississippi River, and sloughing banks.

The bar at the mouth must always be removed by dredging, so long as the current in Old River has no fixed direction. This bar, while varying much as to location and extent, has always been such that it was possible to locate and dredge a suitable channel, and this contains annually about the same cubic yardage.

Heretofore and up to the time of the writer's connection with this work the sloughing banks have been the cause of the greater portion of the dredging in Old River.

In the old bed of the Mississippi, and at distances more or less re-

mote from the present channel, there are large lakes which are filled from the annual floods. As their shores were built up, these lakes from year to year held water at correspondingly greater heads above the low-water level of Old River. The banks were entirely submerged during high water, and were kept thoroughly saturated during low-water periods, and consequently they never attained a stability that would permit their standing at anything but a very flat slope. Mr. Woolley

The banks ultimately attained a height of 25 or 30 ft. above low water in Old River, and sloughing occurred in such masses as to thrust large volumes of material into the channel from the sides. Where the section of the river was quite wide, the large masses of material, sliding down the slopes of the bank, though not able to reach the channel itself, were deposited on the flat portion immediately adjacent thereto. By virtue of their enormous weight, they forced up the bottom of the channel, owing to the unstable nature of the material, thereby forming what are locally termed "mud lumps," completely obstructing the channel which, probably an hour before, had contained from 10 to 12 ft. of water.

There was a time, no doubt, when it would have been impossible or impracticable to obviate this trouble, for during the early stage of this silting-up process the entire old river-bed was a perfect sea of liquid mud, and it was difficult to actually define the real shore lines.

Later, there was a time at which much, and finally practically all, of this sloughing and its attendant evils could have been obviated by simply keeping all these lakes thoroughly drained during the low-water season. The lakes were first systematically drained, by the writer, in the spring of 1894, and have been drained annually from that date to the present, thereby reducing the amount of dredging more than one-half.

The first season the writer was placed in charge of dredging operations in Old River, a thorough survey of the entire river was made, with numerous sections normal to the channel. These sections were connected with an accurate traverse line on top of the bank, and from this a complete contour map was prepared, embracing everything between the tops of the banks. On this map were established what the writer determined should be the fixed channel limits for future dredging operations, all of which received the approval of the District Engineer. These limits were determined with the following points in view—economical maintenance, ease and safety of navigation, tendency to maintain the original location from year to year, economy of first construction, and location where a pilot would naturally seek a channel.

Carefully studying the situation under the above qualifications in the order named, it was considered reasonable to locate such channel limits, with the exception of the channel at the mouth of Old River

Mr. Woolley. (known as Carr's Point Bar), Fig. 20. The latter has had to be located from year to year to conform to conditions in the immediate vicinity which are materially changed with each successive flood of either the Mississippi or the Red River.

The writer has always found, from year to year, remains of the channel dredged the previous year, and, while the channel may be obliterated at one or more points, it is a short, and the most direct, route to connect these disconnected ends, by dredging through the obstructions. After a period of four years the writer is prepared to state that this system of dredging, or fixed channel limits, together with the thorough drainage of the lakes, is to be credited with from 50% to 75% of what was formerly the cost of dredging at this location.

At the time the *Ram* was designed and built, it was intended to dredge out Upper Old River, which is filled almost entirely with a deposit of soft mud, such as the dredge was particularly designed to handle. The tests were made in this material, and showed a capacity somewhat in excess of 300 cu. yds. per hour.

The material handled under ordinary working conditions averaged about 150 cu. yds. per hour. The writer made a test embracing a period of $4\frac{1}{2}$ days on a reef of very compact fine sand, the average for this period being 178 cu. yds. per hour, although the depth of cut across the dredged channel was not uniform, and consequently the work accomplished was somewhat less than the maximum capacity of the dredge, even in this class of material.

This dredge has handled material for $0.025 per cubic yard. The work in some seasons has cost practically double this amount owing to the fact that the dredge was required to remain on the work, with a reduced crew, to redredge the channel should a reversal of the current renew the sand reefs.

All dredging in Old River is to a plane 7 ft. below zero of Barbres Gauge, and to a width of 80 ft. at the bottom, and at the mouth—Carr's Point—to a depth of 8 ft. below zero with a bottom width of 110 ft.

The gauge records throughout a long period show that the direction of the current during the greater part of the low-water period is from the Mississippi toward the Atchafalaya, and the channel at the mouth was designed to conform to this prevailing condition. The outer or Mississippi end of this channel was turned up the Mississippi to receive its supply unimpeded. The inside or Old River end of the channel was trained down Old River, so that it might discharge directly into the deepest water in Old River.

The channel across the bar at the mouth approximates in shape the arc of a circle, with a radius of about 650 ft., and serves the following desirable ends, which are not attained by any dredged channel along the main shore: It provides the shortest route between deep water in the Mississippi and deep water in Old River; it maintains a greater

Mr. Woolley.

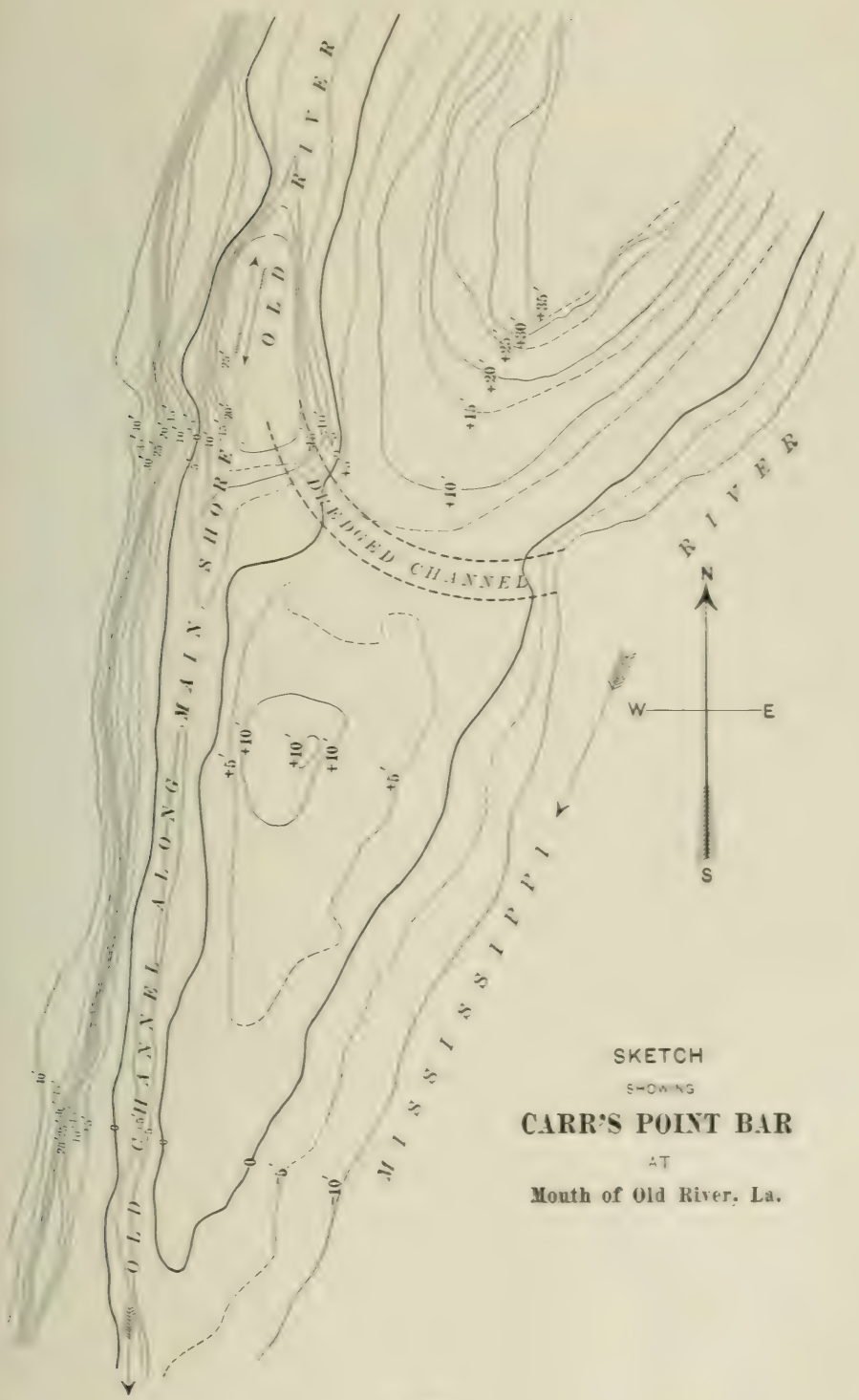


FIG. 20.

Mr. Woolley. head of water in Old River, as it takes its supply from the Mississippi, 4 000 ft. above the mouth of the shore channel, and conducts it directly across the bar into Old River; it effects saving of head, due to the surface slope in. 8 000 ft., and it more nearly maintains itself after being dredged.

The synopsis of the general plan of this dredge given by the author is correct. A copy of the detailed specifications may be found in the annual report of Captain John Millis.*

This dredge has proven to be an exceedingly satisfactory machine, all things considered. The most serious objection that could be raised is to the wooden hull, and the small dimensions for carrying so much machinery, together with the supplies of lines, etc., that are needed for a season's work. The present wooden hull is badly decayed, and it is expected to replace it next year with a steel hull of 25 ft. greater length. All parts of the machinery have proven themselves amply strong and well fitted for the service intended.

Briefly, the general description of the *Ram* and the method of working is as follows:

Hull of decked barge type, 95 ft. long, 27 ft. beam, 6 ft. 6 ins. deep; propelling engines of the modern high pressure, western river type, connected to the usual stern wheel; suction pump 15-in. Edwards centrifugal, three-winged runner, intake in the starboard axis of pump, discharge from bottom of casing and to the rear through pipe suspended below deck to a "Y" near after end of boat, and just forward of spud, the discharge being diverted to port or starboard at will by the "Y;" the out-board discharge is connected to a floating pipe line or to a short section (55 ft.), suspended by shear legs directly from the side of the boat; the pump suction carried by the cutter frame to the base of the cutter head; the cutter head supported at the forward end of the cutter frame, and driven by a 5-in. shaft, made in three sections, connected by claw couplings, all supported by bearings on cutter frame and driven by high pressure engines and sprocket gearing on bow of boat; cutter shaft connected to sprocket gearing by mitre wheels directly on bow of boat, to permit of the cutter frame being raised or lowered through a large vertical arc; cutter frame having a triangular horizontal section, its base being hinged to the bow of boat, its apex or forward end being suspended by wire rope tackle from an A-frame slightly overhanging the bow of the boat, the suspending tackle being connected to hoisting engines on forward deck for raising and lowering to desired depth of cut; swinging chain and cables leading from friction drums on bow of boat, through cutter frame near forward end, to anchors or dead-men on either side of channel; a spud on the axis of the hull 10 ft. from the after end, by means of the swinging cables the cutter is swung

* Report of the Chief of Engineers, U. S. A., 1894, page 2979, Appendix XX.

back and forth across the channel to be cut, with the spud as a center, and the distance between spud and cutter as a radius. Mr. Woolley.

For the work in Old River, with the strong current which usually prevails, it was found very difficult to use the pontoons, even where the depths outside the channel were sufficient, and there were many places where they could not be floated. It was then decided to suspend as great a length of discharge pipe as possible from the side of the boat, and discharge directly overboard. By keeping the dredge well to the side of the channel, on which the discharge was being placed, taking the full swing in the opposite direction, it was found that the material could be placed 60 ft. away, or on top of the flat shelf outside of the dredged channel. With the class of material encountered in Old River, it was found that a very small percentage of discharge returned to the channel. The ease of manipulation much more than outweighed the return of material to the cut, as compared with the stoppage and loss of time due to buckling and fouling of the pontoon line in shallow water.

In cutting the channel across the bar at the mouth, the discharge was always placed on the down-stream side, and although the material at this point is largely sand and gravel, a comparatively small amount of it returned to the cut.

The bar at the mouth had been cut through when 500 ft. of the proposed channel was high and dry above the water surface. In cutting through a dry bar the cutter head is lowered to the desired depth of channel, and worked forward just as fast as the material sloughs down the face of the cut. Of course, no material standing above the water surface could be cut through and not slough down the face of the cut, without much delay from moving back to clean up material that might have fallen behind the cutter head, unless blasting or other artificial means were used to throw down the face of the cut slightly in advance of the cutter. For this same reason no great depth of cut can be taken in very stiff clay without a considerable fouling of the channel behind the cutter head.

From the foregoing it will be quite evident that the depth of a single cut, within certain limits, has to be determined by the class of material being handled.

The dredge is always worked down stream, and the greater the velocity of the current, the greater the necessity for so doing, to prevent the fouling of the channel behind the dredge.

For all classes of material encountered in Old River, it is found that the channel behind the dredge does not differ less than 0.5, nor more than 1 ft. from the depth at which the cutter head is being worked.

When dredging in pure sand, the depth of clear channel behind the dredge and the depth of the cutter head varies from $\frac{3}{4}$ to $1\frac{1}{2}$ ft.,

Mr. Woolley. and much of this difference the writer attributes to material running from the sides. The *Ram* was fitted and designed to work with a walking or push spud, to be operated by steam, the spud to be raised and the foot set forward again after the dredge had been fed into the cut through the maximum movement permitted by the spud. This was never successful. The pushing gear was weak and defective, and the dredge was always thrown back from the face of the cut when the spud was being raised.

The spud is now used only to hold the dredge in place. When the spud is raised, the dredge is fed forward by the propelling engine. Under ordinary working conditions and depth of cut, the cutter head is swung across and back again over the desired width of cut, the spud is raised and the dredge forced ahead by propelling engines. The usual forward movement is from 18 to 24 ins. each time, and at about four or five minutes apart.

A careful study of the daily log of the *Ram* during the working season, since she was placed in commission, shows that seven and one-half out of every ten hours are actually devoted to dredging. This loss of but 25% of the actual working time is for all causes—minor repairs, moving swing cables forward to dead-men or shore anchorages, lowering swing lines for passing boats, raising cutter to remove débris or lubricate shaft bearings, and moving dredge to fuel barge for coaling bunkers when working without the service of a tender.

After four years' experience with this particular dredge, the writer is of the opinion that a machine of this type, which can devote 75% of its working time to actual dredging, is quite up to, if not above, the average. A dredge specially built to work in the open Mississippi, and not subject to many of the stoppages hereinbefore mentioned, should be able to average 8 to 9 hours' actual work out of every possible 10, or 19 to 21 out of every 24 hours.

In 1896 the Mississippi River Commission directed that the *Ram*, in addition to its duties at Old River, should care for the needs of navigation at Ford's Crossing, none of the other dredges built for use in the Mississippi being available for that locality.

Ford's Crossing (Fig. 21) is in the Mississippi River, about eleven miles below Natchez, Miss. This bar does not annually become a serious obstruction to navigation. In only about one of every three years has the depth of water been less than 7 to 8 ft.

In 1896 the greatest channel depth across this bar was but 5.4 ft. The crossing was rather abrupt, being at an angle of about 45° to the axis of river, and so crooked as to offer serious obstruction to the navigation of boats drawing even less than 5 ft. A survey was made and a contour map of the bar prepared. The channel was located by the most direct route from the upper to the lower pool, parallel to the thread of the current and practically over the nearest approach to any

existing channel. Numerous floats were run from the upper to the lower pool, their course being accurately located on the contour map. All available data were taken and noted in order to lay the proposed channel so as to coincide, as nearly as possible, with that which Nature would seem to indicate as her choice. The total length of channel dredged was 2 400 ft., to a depth of 12 ft. below extreme low water, with a bottom width of 100 ft. A navigable channel of 9.5 to 10.5 ft. was maintained during the season by going over the cut twice. The dredge was entirely inadequate to cope successfully with a bar of this magnitude. The conditions were unfavorable for a test of a dredge of this particular type and small capacity for bars in the Mississippi. Mr. Woodley.

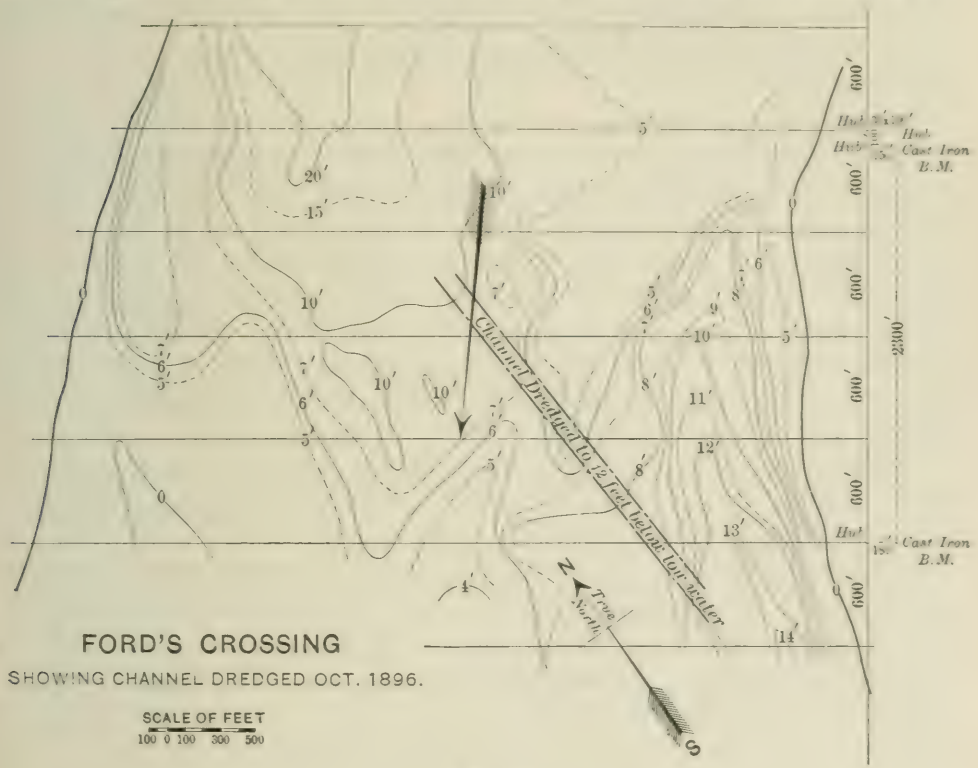


Fig. 21.

The river was falling during the first cut and rising during the second. There was a much greater back-fill and running of sand from the sides during a rising stage. The effects of dredging seemed to be pretty well obliterated five days behind the dredge on a rising river. With a falling river the channel behind the dredge maintained itself much better than had been expected. These conditions may apply generally to this and many bars in the Mississippi, or they may be as erratic as some natural locations of channels across bars.

The material encountered at Ford's Crossing was sand, mixed with about 1% of gravel. The sand was much coarser than that encountered at the mouth of Red River, and worked much easier, the dredge

Mr. Woolley. handling about 20% more per hour than at Red River, the average of solid matter being about 12 per cent.

The writer's experience in dredging large sand bars in the Mississippi River has been limited to this work at Ford's Crossing, with this comparatively small sand bar and small dredge; beside which the larger bars and larger dredges, in the upper Mississippi, are about as 1 to 10.

However, the experience gained at Ford's Crossing has, in the opinion of the writer, determined that the prerequisites to the successful dredging of a navigable channel through the bars in the Mississippi River are:

First.—Machinery of large capacity, able to burst through a bar in the shortest practicable time in order to secure the scouring action of the current before the channel behind the dredge shall have deteriorated.

Second.—A location of the proposed channel to conform, as nearly as can be determined, to Nature's choice, from which the slightest deviation will develop insurmountable obstacles, resulting in expensive and useless labor.

Before having had actual experience in dredging a bar in the open Mississippi, it occurred to the writer that placing the discharge 1 000 ft. away from the dredge was rather excessive. The experience at Ford's Crossing gave every indication that if the discharge had been placed 300 ft. away from the channel the distance would have been ample.

The sand on the bar at Ford's Crossing runs very much more freely than anything found at the mouth of Old River, owing to the larger grains composing the former.

It can be readily understood that the bars in the upper Mississippi are composed of a material that is subject to greater movement than that composing the bars in the lower Mississippi. The writer does not wish to be considered as implying that the discharge distance of 1 000 ft. has been adopted without careful tests, or merely because these dredges were fitted with this amount of discharge pipe.

The writer has seen no published results of tests to determine the practicability or economy of a shorter discharge distance than 1 000 ft., and mentions this with a view of bringing forth anything that may be available on this interesting point.

The following memoranda may be of interest as relating to the efficiency of the several parts of this machine, the alterations which have been made, together with others that it is proposed to make in order to place the dredge in thorough condition before the season 1899.

The cutter head has proven to be amply strong for all kinds of work. Many piles, sunken saw logs, steamboat wrecks, heavy oak checkposts in old sunken coal barges have been encountered and disposed of, and not a single blade on the cutter head has been broken.

Mr. Woolley.

The cutter head could be designed with larger openings for certain classes of material, which would increase very much the efficiency of the machine.

For the various classes of material, from stiff clay to sand, and including all kinds of débris, the present design of cutter head is well adapted.

The 15-in. Edwards pump has given great satisfaction, is comparatively easy of repair, has worn extremely well and is admirably adapted to this kind of work.

There is but one slight objection to the details of this pump : For the high speed required, the writer has found need for larger thrust bearings. With some minor alterations, and with water-jets, the writer has been enabled to run the pump at 260 revolutions per minute for eleven consecutive hours, with a vacuum of from 4 to 27 ins., without undue heating.

It is proposed to increase the bearing area of the thrust rings about 50% as soon as it becomes necessary to put in new rings.

The compound engines were built by the Bucyrus Company after their own pattern; high pressure cylinder $14\frac{1}{2}$-in. by 20-in., low pressure cylinder 26-in. by 20-in., placed on a rigid, hollow-girder bed-frame.

The engines are driven at 130 revolutions per minute, and develop under ordinary working conditions, with 100 lbs. boiler pressure, from 170 to 190 H.-P. The engine and pump are connected by belt; the revolutions of pump and engines having a ratio of 2 to 1.

The first season's work developed the fact that the engine shaft was too light to carry the heavy driving pulley without heating its bearings, the shaft becoming flexible when running with full load and thus causing great vibration throughout the boat.

The shaft was made as large as housing could accommodate, the hollow cast-iron caps forming the top of the bearings were tapped and a stream of cold water constantly circulated through them. No further trouble was had from heated bearings.

The present wooden hull has proven entirely too small for the convenient manipulation of the machinery, and it is proposed to replace it with one of steel. The new steel hull will be 25 ft. longer, will contain ballast tanks on either side, to enable a greater length of discharge pipe to be suspended from the side of the boat, and at the same time to allow the boat to work on an even keel, without the necessity of constantly shifting fuel from bunker to bunker for ballast, as is now done.

The propelling engines are capable of driving the dredge and a tow, consisting of a large fuel barge, with fuel for an entire season's work, as well as all pontoons and other necessary equipment, upstream at the rate of 6 miles per hour.

Mr. Haupt. L. M. HAUPT, M. Am. Soc. C. E.—This valuable paper is so comprehensive as to open a wide field to discussion, and one cannot do it justice in a brief contribution, with limited time at command.

There are several points to which attention is directed. Before submitting them, however, the writer would remark, *en passant*, that at the Annual Convention of 1892 he called attention to the fact that, unless the discharge of sediment at the delta was equal to or greater than that carried into the bed of the river by its tributaries and caving banks, the bed must inevitably rise. This proposition appears to be confirmed by the physical description of the river, showing a diminution in slope and absence of bars in the lower reaches, the coarser gravels being deposited higher up, and only the fine mud being carried to sea.

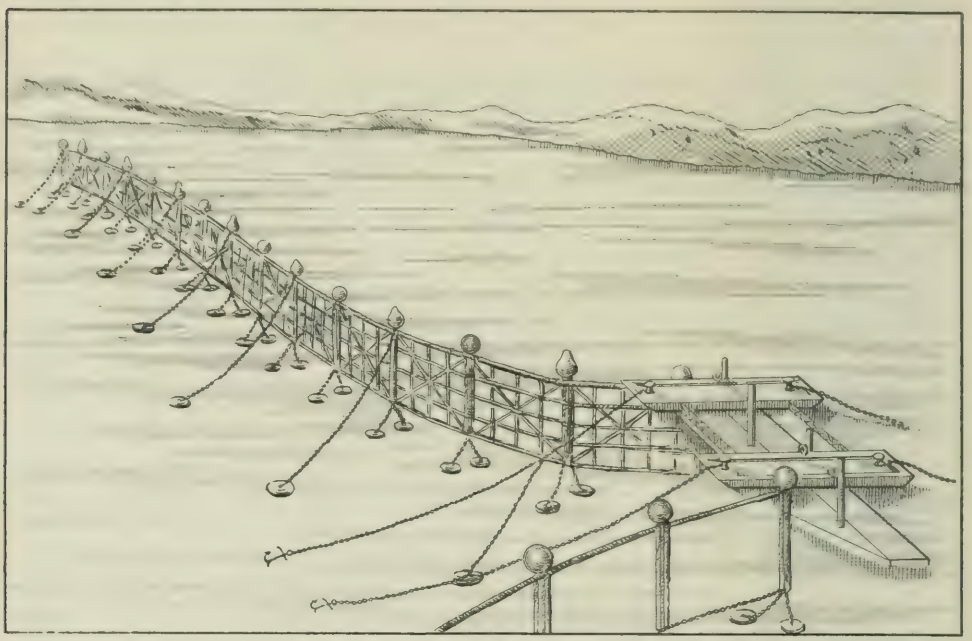


FIG. 22.

Under the subdivision of current deflectors, the author states (page 233): "In practice, it is a well-known fact that a barge anchored in shallow water will very soon cause the sediment to deposit." Etc.

On the contrary, it was found during the building of the St. Louis Bridge that the scows placed around the abutments produced a scour under them.

The writer also observed that the vessels lying alongside the bulkhead of the Atlantic Refining Company's wharf in the Schuylkill River, Pa., where the currents are sluggish, caused a well-defined hole to be scoured under their berths, while a ridge rising from 11 to to 13 ft. formed outside near the axis of the stream. The low-water depths along the bulkhead exceed 20 ft. Again, the writer made an

experiment on the Five Mile Bar, with current deflectors hinged to piles to test their efficiency, and within two tides the depths were increased from 9 to over 18 ft., M. L. W. Mr. Haupt

The writer is, therefore, at a loss to understand the local or other conditions that have apparently produced such opposite results from current action due to vertical deflection.

The description of the canvas curtains is akin to the devices patented by the writer in 1886, and a reference to Fig. 22 will suffice to show the idea.

Again, the 5th class was clearly anticipated by a device tried by William P. Craighill, Past-President, Am. Soc. C. E., on the lower Susquehanna with very good results. It consisted of two or more boats or rafts, with vertical sheet piling suspended between them, to be used in trials for the proper location of a more permanent dike.*

The 8th class is also anticipated in the Brownlow weeds, so successfully tried on the Missouri River in 1878 for the purpose of contracting the river by creating artificial bars by means of these permeable dikes,† which proved so cheap and effective, as compared with the revetments.

The following extracts ‡ may serve to supplement this interesting digest by the author, as it describes devices to which no reference appear to be made:

"*History of Vertical Deflectors.*—Probably the earliest attempt made to apply a vertical deflection of the surface velocity upon the bottom was patented by General G. T. Beauregard in 1853. The device (Fig. 23) consisted of a large, rectangular wooden tube, open at both ends and having a wooden diaphragm or inclined plane extending from the upper edge at the inner end to near the lower edge at the outer, for the purpose of reflecting the surface current downward to act upon the bar. 'The excavator is to be anchored at the inner edge of the bar by weighting down the top.' Thus it appears it had no means of locomotion, the anchorage was very deficient, there was no assurance that it would always remain 'right side up,' nor that it would not be submerged by the scour at its ends and sides. It would probably soon have been dashed to pieces, capsized or buried. There is no evidence that it was ever tested.

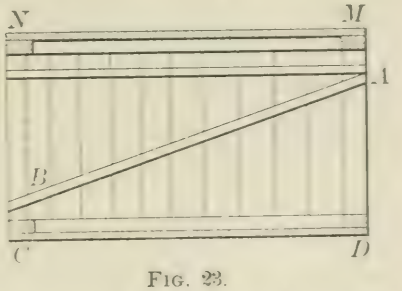


FIG. 23.

"In 1860 another attempt was made, by Robert Gamble, Jr., to apply this principle in a more practical device. It consisted of a

* Report of Chief of Engineers, 1867, pp. 419–427.

† Reports of Chief of Engineers, 1878, p. 654, etc.

‡ These quotations are taken from a paper by the writer and may be found in the Proceedings of the Engineers' Club of Philadelphia (Vol. vi, 1886–1888, p. 73, etc.), but Figs. 23 to 27 are added and there are also some other slight changes to make the matter more complete.

Mr. Haupt. floating barge (Fig. 24), having a false bottom hinged near the up-stream edge. The bottom was composed of two leaves, the lower being kept horizontal as it was pressed down by four spuds, while the upper was made to flare up stream to deflect the current under the boat, and so wash out the bar upon which it had grounded. The action in this case, it will be seen, was limited and temporary, and did not differ, in kind, from that of an ordinary scow, which, being anchored over a shoal, will cause the removal of the sand under it for a depth nearly equal to its draught.

"A third device was that proposed by J. Grant, a Mississippi River pilot, in 1876, who tried to remove bars by placing a barge, having heavy scantling suspended on its side, athwart the current. Attached to the bottom of the scantling there were flaring shields, 3¼ ft. wide, extending the whole length of the boat, 80 ft., by which the currents near the bottom were deflected downward. This machine was placed near the upper end of the shoal to be removed, and the aprons lowered to the required depth, when it was found that the current was forced under it with increased velocity, and the material was quickly excavated, thus allowing the barge to float down stream, guided by a tug, and shearing off the top of the bar as it went. 'When a deepening of about 6 ins. had been effected over the whole area required, the apron was lowered still deeper, thus shaving off the shoal by successive cuts.' By this means a deepening of about 2 ft. was effected, but when operations ceased, the bar reformed.* This serves merely to demonstrate the correctness of the principle of vertical deflection, for in this instance the feeblest currents or those near the bottom were the ones applied, and that only for a very limited depth. There was the fatal defect of not providing for a permanent change of regimen at that section, for no mere change of mould without a corresponding change in velocity will produce a permanent channel."

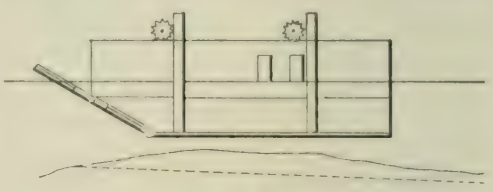


FIG. 24.

In 1880 Taylor patented an intricate device of a weighted triangular box, a floating barge with movable truck, a submerged pilot wheel, an articulated brace or leg, and other complicated parts.

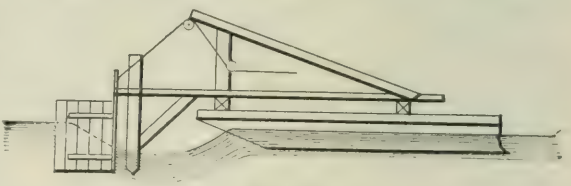


FIG. 25.

Koon's device, of 1883, consisted of a series of deflecting plates set in a box, or frame, having legs which rested upon the bar, but the writer has never seen any record of a trial having been made.

"*Economy of Current Deflectors.*—As to the economy of this method of dredging as compared with the mechanical devices, reference is made to a crude and limited construction (Fig. 25) used on the Garonne in France; where, notwithstanding its imperfections, it was found quite efficient. The account says: 'An economical manner of removing the materials of the bed of a river consists in the application

*The Mississippi Jetties. Corthell, p. 216.

of movable dams attached to a boat. These dams offer a resistance to the flow of the water, and, according to the greater or less depression given to them, they either divert the whole effort of the stream against any particular object, or by simply contracting the water-way they augment its velocity, especially at the bottom. Wherever applicable this system has been found to diminish the expense by about nine-tenths.' Mr. Haupt.

" 'The machine which was employed on the Garonne removed about sixty yards of sand and clay per day, at an expense of about 2½d. (5 cents) per cubic yard.'

" (Extract from the '*Aide Memoire*,' Vol. 3, p. 273).

" It appears that this dam was a kind of vertical screen attached to the up-stream end of a scow to contract the waterway."

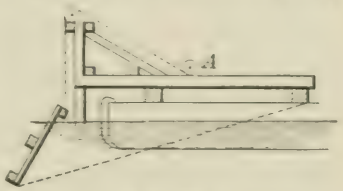


FIG. 26.

The "Kingston" deflectors (Fig. 26) as applied on the Hooghly River, India, cut from 6 ins. to 2 ft. through a bar 300 ft. long in two hours at low water.*

The experience of the writer inclines him to the belief that current deflectors are very effective auxiliaries to channel improvements, but they should be accompanied by permanent regulating works to maintain the benefits which they produce and augment. Fig. 27 shows a current deflector designed by the writer some years ago.

Under the subdivision of water jets, etc., the writer is impressed with the failure to refer to the pioneer and basic patent for the system of removing spoils from the bed of a river or bay by means of rotary cutters with inward delivery, centrifugal pumps and flexible distributing pipes, as issued to Mr. A. B. Bowers, C. E., of San Francisco, and which has been applied so extensively in various modified forms on rivers and harbors, to the great benefit of commerce. The claims of the patentee have been uniformly sustained by the United States courts, and the inventor's time is consumed in efforts to protect his interests, the history of which reads like a romance. Active opposition to an invention is, however, the best evidence of its merit. These dredges were used at St. Louis, on the Mississippi; on the Chicago drainage canal; at Seattle, Tacoma, Alameda Bay and, in modified forms, on the Atlantic and Gulf coasts, with great effect.

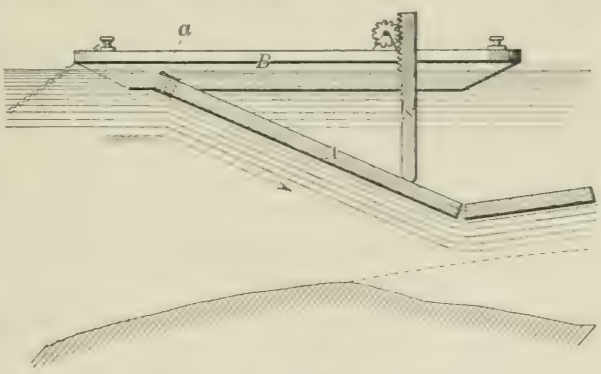


FIG. 27.

* For details and drawings see *Scientific American*. November 24. 1888. p. 326.

Mr. Haupt. Under Section 7 the author says: "The removal of bars by means of blasting has been tried with unsatisfactory results." If this statement is understood as being limited to the experience on the sand bars of the Mississippi River it may doubtless be correct, otherwise it is open to criticism, since other instances of the successful removal of bars may be cited, one of the most conspicuous being that of the Brunswick Bar, Georgia, where the depth was increased from 17 to 24 ft. by the explosion of dynamite aided for the last foot or more by devices for smoothing the bottom, and the results appear to be fairly permanent, while the cost was a mere fraction of that which would have been required had jetties been built.

This excellent paper is, however, too extensive to be discussed in detail, and the writer believes that the temporary improvement of the Mississippi, at low-water stages, by dredging is doubtless the most expedient method of maintaining a navigable channel, especially with the hydraulic dredges which have been so well developed under the Commission. The capacity of these dredges reached, in one case, 2 553 cu. yds. per hour, an achievement which would have been regarded as utopian but a few years ago, when the average was 1 000 cu. yds. per day.

The adoption of the dredging system, for the improvement of the river, marks a radical departure from that used in the earlier stages of its development, when these modern facilities were not available; but, as the author states, the making of each cut involves a special problem, and the results are not permanent, but must be repeated at each low-water stage. That this may be done economically it should be remembered that a lateral displacement of material, especially if placed on the down-stream edge of the cut, when it is not parallel with the axis of the current, will produce the best results, both by concentration and reaction. A change of form in the wetted perimeter without an increase in the area of the section is what is desired; hence, in general, a longitudinal movement of material, unless it be intended to produce uniformity of section and flow, would prove injurious.

In conclusion the writer submits another brief extract from his paper on the improvement of rivers by regulation works.*

"With reference to the improvement of the Mississippi River, the plan of the Commission as stated by Mr. Jas. B. Eads, C. E., was to rectify 'the high-water channel by the ultimate narrowing of the wide places as the only method by which a deep and uniform low-water channel can be permanently secured.' 'A uniform current will result; the caving of the banks will be practically arrested, and the sooner these wide places are corrected, the less will the improvement of the river cost.'†

"In a stream, the improvement of which may cost over $100 000 000, it is of great importance that the most economical and permanently effective devices be used.

* For further data see also article on Dredging, in Johnson's Encyclopædia by L. M. Haupt.

† N. B.—The policy of the Commission having changed, Captain Eads withdrew.

"It is evident that the velocity of a stream is affected by the amount or quantity of water, and the area of its section. If the latter be made larger for a given quantity, the velocity will be reduced; if smaller, increased; or, if the quantity passing a given area be increased or diminished, the velocity must vary in the same ratio. Mr. Haupt.

"It is also apparent from the preceding quotations that in the opinion of the most experienced hydraulic engineers, the only forces that can be applied with success to secure permanent improvements are those of tidal and river currents. Hence that system which most readily controls and regulates the currents will give the best permanent results in curbing the stream."

A. W. Robinson, M. Am. Soc. C. E.—This paper is valuable and timely as a historical record of a most remarkable work. The author's familiarity with the Mississippi has enabled him to describe the peculiar conditions which there exist, and to make clear the reasons and considerations which have governed the development of a unique system of dredging. In the low-water season on the Mississippi events follow one another rapidly. A dredge that can make an available channel across a newly developed shoal in a few hours and permit navigation to be resumed can fairly be called a unique production. Mr. Robinson.

The author has described the first efforts made in this direction and the rapid development of a distinctive type of dredge from the *Alpha*, of 1892, to the machines now being built. Four of the dredges described in the paper were built by the Bucyrus Steam Shovel and Dredge Co., namely, the *Ram* and the *Gamma*, both from the writer's designs, and Nos. 3 and 4, as described on page 293, from Major Handbury's designs.

In the construction of the earlier dredges there was some difference of opinion as to the relative value of different features. The committee having the matter in charge determined to construct different dredges to test the most promising designs. For instance: It was thought by some that mechanical agitators were indispensable for efficient work, and the good work done elsewhere with rotary cutters, etc., was cited; on the other hand, there were those who contended (the late Colonel Henry Flad and the writer, among the number) that, while mechanical agitators were useful and even indispensable under certain conditions, they were unnecessary in the free, alluvial sand of the Mississippi River. Experience has proved that the cutter appliances only add complication and liability to delay, and that better results are obtained with water-jets, or in fact with no agitation at all. It is only necessary to bring the mouth of a suction pipe into proximity with the sand, and to feed it regularly, in order to obtain all the sand that the pipe is capable of carrying. If the open end of a plain suction pipe is fed into a bank of free sand, it is always possible to choke it by taking more sand than the water will carry. Observation of the early work of these dredges shows that there was no difficulty in getting sand into the pipe in any quantity, and that the main thing was to put it through in large quanti-

Mr. Robinson. ties without choking and without sinking the pipe. This question of mechanical cutters *versus* water-jets is, therefore, one which should be determined entirely by the conditions to be met and the kind of material to be dredged. In free sand neither are necessary. In sandy alluvium, soft mud, or stratified deposits easily separated, the water-jets will be sufficient to break up the bank so that it will offer no resistance to pumping. In harder material, requiring cutting and disintegration, mechanical force must be applied as well as hydraulic force. These and other questions have been settled by the experience gained, and while all the dredges, as a series, are remarkably successful, the later ones are naturally more perfect and efficient.

The first floating discharge pipes made had insufficient buoyancy. It was thought that more than 25% of sand never could be carried in so large a pipe, but later it was found necessary to provide a margin of buoyancy even with the pipe solidly full of sand, a condition which sometimes occurred. Owing to the great size and weight of these discharge pipes, the general practice has been to carry them partially submerged. This caused some inconvenience in fitting on the rubber connecting sleeves, but made it possible to use smaller pontoons. In the pontoons of the *Gamma* each section consisted of four cylindrical pipes clamped to one length of discharge pipe 50 ft. long. Four clamp frames on each length served to bind the whole rigidly together into one pontoon, without using any riveted fastenings or making any holes in the floats. This construction gives great stability against overturning, and also admits of turning, removing, getting at or replacing any of the floats, without difficulty. It is also cheap and easy of shipment. The side-floats of the *Gamma* were made a little smaller in diameter than the main central pipe. Experience showed that it would have been better to provide more buoyancy by making the side pipes at least of the same diameter as the main discharge pipe.

The pontoons of the *Beta* were rounded on the bottom, with the object of diminishing the resistance to the current when the pipe-line is deflected or lying across the current. They were, however, deficient in stability and were liable to capsize. In dredges Nos. 3 and 4, Major Handbury has treated the pipe-line question in an entirely different manner. Instead of a floating pipe 1 000 ft. long, deflected across the current and subject to its resistance, he proposes to use a pipe 500 ft. long, and carry it straight out laterally, at right angles to the current if necessary. This is accomplished by mounting the pipe on a series of boat-shaped pontoons with swivel supports on each. The pontoons are sharp at bow and stern, are each provided with a rudder, and can be so steered and set that they will carry the pipe out, at any desired angle from the dredge, in a rapid current, without any deflecting lines or anchorage whatever.

The enormous output made as a test record by the *Beta*, in spite of

Mr. Robinson.

many and serious mechanical defects, is a proof of what sand and water will do when brought within the range of the suction of 2 400 H.-P. The cutter appliances of the *Beta*, consisting of six vertical cylindrical steel cutters 6 ft. x 6 ft., operated by 26 bevel and other gears, contributed in nowise to the result, except possibly as a strainer to admit water and thus prevent choking, for only the extreme lower end of the cutters ever came in contact with the sand.

As a result of the experience gained the entire cutter system of the *Beta* has been discarded, and also the auxiliary engines and winding gear, and the dredge fitted with a water-jet suction and anchor winches, practically on the design of those used on the *Gamma*.

In the matter of boilers for these dredges there is some difference of opinion. Three of the dredges are fitted with Heine water-tube boilers, and the remainder with externally fired, cylindrical, return-flue boilers, set in brick-lined casings—otherwise known as "Mississippi River" type. The *Ram* has cylindrical, internally fired, tubular boilers. The old river type of boiler has remained unchanged for many years, and steamboat men have become so accustomed to it that its use is second nature to them, and they naturally regard the modern mechanical refinement of a water-tube boiler with suspicion and distrust. The old-fashioned boiler answers the purpose well enough with unskilled labor and in muddy water, and for these two reasons, as well as for the fact that it is cheaper in first cost, it has the advantage.

The result of the experience gained with the six previous dredges may be expected to be found embodied in the latest dredge which is now to be built for the Mississippi River Commission.

The *Beta*, *Delta* and *Gamma* were ordered by the Mississippi River Commission from Lindon W. Bates, the New York Dredging Company and the Bucyrus Steam Shovel and Dredge Co., respectively, for the express purpose of demonstrating whatever merit there may have been in the three designs. The commercial results shown by the tests of these three dredges may be indicated briefly by the following figures taken from the paper:

Dredge.	First cost.	Operating expenses per day.	Test Output.	
			Average cubic yards per hour.	Cubic yards per horse-power per hour.
Beta	$218 162	$221.63	4 920	2.60
Delta	124 940	111.76	1 829	1.62
Gamma	85 530	100.51	1 523	3.28

The Mississippi River Commission is now in possession of many data and results of practical work, and the dredge recently called for in

Mr. Robinson. the specifications issued June 21st, 1898, may be assumed to be based on careful study of the performance of those already in service.

The following points are noted in these specifications :

(1) Boilers of the river steamboat type are retained.

(2) The main pumps and engines are of the *Gamma* type, improved in detail.

(3) The feeding and hauling winches are practically of the *Gamma* type.

(4) The dredge is to be self-propelling.

(5) The discharge pipe is to be 500 ft. long, and carried on top of pontoons instead of being partially submerged.

The results of the comparative tests that were made, as given in the paper, cannot fail to be of interest. Such tests could only be made by the Government, as they involved expense that no contractor or private corporation would care to incur. While, in the main, the tests appear to be reliable and accurate, some of the statements are open to question : For instance, on page 285, in the tests of the *Epsilon*, and in the third and seventh tests the efficiency is given as 83.6 and 91.8%, respectively. As this efficiency is the percentage of the foot-pounds of work done by the pump to the foot-pounds of work represented by the indicator diagrams of the engine, it represents the combined efficiency of both pump and engine. The power consumed in internal friction of the engines is not given, but, assuming it to be 10%, this would make the net efficiency of the pump 92.9 and 102%, for the tests in question. Needless to say, such a result is impossible. Something appears to be wrong in the velocity of discharge. In Test No. 6, with 738.3 I. H.-P., the velocity is 16.6 ft. per second, and in Test No. 7, with 717.1 I. H.-P., the velocity is 18.3 ft. There is not sufficient information as to the details of the observations to enable an intelligent criticism of the causes of these discrepancies to be made. Slight errors in the observations or instruments and changes in conditions might cause great variation. The velocity of discharge appears to have been computed on the assumption that the entire cross-section of the pipe was moving, whereas this may or may not have been true.

Mr. Ockerson. J. A. Ockerson, M. Am. Soc. C. E.—The author is highly gratified with the scope of the discussion brought out by the paper. The records of the experiences of different engineers working in different fields on similar problems are exceedingly valuable to those who come after them. The paper has at least served a good purpose in bringing out these individual expressions.

In reviewing the discussion it may be well to state that the author did not intend to exhaust the subject in the brief space of a single paper, and purposely omitted descriptions of many channel improvement devices that have come to his notice. It may also be well to

state that he aimed to limit himself to devices proposed for, or used on, the Mississippi River. Mr. Ockerson.

With these explanations Professor Haupt will understand why the author failed to "refer to the pioneer and basic patent," etc., and also omitted the mention of many devices, several of which have been described by Professor Haupt.

As a further illustration of the silting up due to the retarding influence caused by contact with the bottoms of barges and boats, it may be mentioned that where large fleets of plant are tied to the banks for considerable periods of time the shoaling frequently becomes so serious as to necessitate their removal to some new site.

A dredge and its fuel barge left in the dredged cut has been known to fill it up to such an extent as to obliterate the cut in a single day.

Mr. Edward Flad in his discussion disputes a statement made by the writer with regard to the results obtained with jets and mechanical agitators.

The following extracts from the official reports may throw some further light on the question. The engineer officer in charge says:

"The *Zeta* differs from the *Epsilon* in the use of a mechanical agitator in place of a water-jet agitator. On account of the numerous break-downs on this dredge, which did not amount to enough to cause a rejection, but did indicate her inferiority as a working machine, it was finally determined to add a water-jet agitator, and the necessary machinery for the same has been ordered."

The mechanical engineer under whose supervision the dredges were built, says:

"The dredge *Zeta* which was fitted with a mechanical agitator, did not pass the ordeal of testing so satisfactorily. * * * The record of these tests is a record of stoppages to put in new shearing pins. * * * The performance of this agitator was considered so unsatisfactory that it was deemed inexpedient to take the field relying solely upon its help to loosen material for the pump. * * * The results of the tests of the *Zeta* (which is a duplicate of the *Epsilon* with the exception of the agitators) show a decided diminution in the quantity of sand delivered, which may be accounted for by the difference in the efficiency of the agitators; the average of five tests with the *Zeta* being 1 364 cu. yds., and of the *Epsilon* 2 553 cu. yds., of sand per hour."

The superintendent of dredges, under whose personal supervision all the tests were made, says:

"It was found, while making the tests with the *Epsilon*, that the best capacity test was obtained when pulling ahead at the rate of about 9 ft. per minute. When the test of the *Zeta* was commenced it was found impossible to pull the dredge ahead at that rate, owing to constant breaking of agitator shearing pins. It was therefore necessary to reduce the speed of advance down to less than two-thirds of that of the *Epsilon*, in order to have the shearing pins last long enough to make the barge test. * * * It was not thought advisable to increase the size of these pins beyond $1\frac{3}{8}$ ins., as it might

Mr. Ockerson.

break either the bell-crank or some other part of the machinery which was becoming badly racked. * * * It was therefore decided to purchase a jet pump and place it on board in connection with this agitator for use in the field, as the capacity of the dredge was cut down nearly one-half, owing to the mechanical cutter."

The above reports were written before the dredges had been used in actual channel dredging. Since then, they have seen some three months' service in actual work, which has verified the above conclusions as to the mechanical agitator.

Mr. Flad also states that the writer is mistaken in saying of the agitator, "It will therefore be taken out, and jets will be put in." Jets have been put in, and the mechanical agitator has been taken out and is now lying on the "plunder barge." This, however, was not done until it was tried in average river sand under ordinary working conditions and found to be deficient as an agitator.

With this slight exception, the dredges *Epsilon* and *Zeta* have proven in actual work to be excellent dredging machines.

Mr. Robinson's discussion is of special interest owing to his wide experience as a designer and builder of dredges. He seems to question the reliability of some results of the tests that have been described. It should be borne in mind that in these tests, the dredges are put to their best paces for a short interval of time, and therefore may be expected to develop an unusual degree of efficiency.

There is one feature of the tests as recorded in the tabulations which should be explained. The test barge is 1 000 ft. away from the pump when the tests are made. The indicator cards, revolutions of pump and other items pertaining to the mechanical performance of the dredge, used in deducing the results are determined at the time when the valve in the barge is open. The distance between the two being so great, and the time interval so short, the conditions noted at the dredge may not reach the barge until after the valve is closed. As the material is continually changing in both character and quantity this may account for some of the apparent discrepancies. The reason the observations at the dredge and at the test barge are not made so as to cover the same conditions is because the operators at the dredge are not permitted to know when the valve in the barge is to be opened. In this way the natural disposition to have the pipes filled to their limit with sand, just as the test is made, is in a measure obviated.

It must not be supposed, however, that equal indicated horsepower will give equal velocity of discharge, as it is no uncommon thing for the suction head to change 50% in a brief interval of time. This is due to the variations in the character of material, which at times does not readily enter the suction. Considerable power is therefore used up at the suction side of the pump.

Mr. Robinson says: Mr. Ockerson.

"Something appears to be wrong in the velocity of discharge. In Test No. 6, with 738.3 I. H.-P., the velocity is 16.6 ft. per second, and in Test No. 7, with 717.1 I. H.-P., the velocity is 18.3 ft."

He failed to note that in the first case the suction head was 17.0 ft. and in the second case 14.7 ft., while the delivery head remained the same in the two cases.

The velocity of discharge is "computed on the assumption that the entire cross-section of the" discharge pipe flows full during the test.

The total volume found in the test barge divided by the number of seconds occupied in filling the barge, gives the volume per second. This volume, divided by the area of the discharge pipe, gives the velocity of flow per second. With the pump running uniformly at the normal speed, and with the high velocity of discharge attained, it does not seem probable that the section of the pipe is materially reduced by a deposit of sand at rest on the bottom.

Under abnormal conditions, such as running the pump at a very high speed so as to cause a slip, there would be a very low discharge velocity. This would very seldom happen with the large sized pumps in use.

The observations of all kinds, recorded during the tests, were made with great care and are believed to be reliable as far as they go. If there are discrepancies in the results that cannot be explained, it should be remembered that, in dealing with steam and water, the laws governing their expansion and flow have not as yet been fully and satisfactorily defined.

Mr. Mitchell says:

"Occasionally a dredge-cut, without other work to maintain it, may deepen and improve, and become the permanent channel for that low-water season, but more often it will require a repetition of the work of the dredge," etc.

This is contrary to the experience on the lower river. The cases where dredging has to be repeated during the same low-water season have thus far been exceptional. When there are considerable oscillations of stage, the cuts sometimes fill.

During the present low-water season there have been several oscillations ranging from 2 to 10 ft. or more. In one case a cut that had been completed was found to be filled up by the rise in the river, and preparations were made to open it again when the river fell. A survey at that time developed the fact that the material which had filled loosely into the cut was all washed out again.

The work of the present season coincides with the experience of previous work, to the effect that a cut once well opened will generally remain in good condition during the low-water season, and, perhaps, even longer. The exceptions are probably due to faulty

Mr. Ockerson. locations, or unusual changes in the local regimen or in the stage of the river.

The use of pontoons for sustaining the discharge pipes has some advantages over the floating discharge pipes, the chief of which is to be found in the facility with which the successive lengths can be coupled, as the pipe lies entirely above water. It can, perhaps, also be deflected more readily than the floating pipe line. In practice there has been very little trouble with the latter. As far as simplicity and facility of handling in towing are concerned, the floating pipes seem to be better than the pontoons.

Mr. Woolley's description of the work done at the mouth of the Red River is a very interesting account of a successful piece of hydraulic dredging.

In regard to the length of discharge pipe necessary on the bars of the Mississippi River, Mr. Woolley questions the necessity for using a discharge 1 000 ft. long. There are many cases where a length of 500 ft., or even less, is ample, and in such cases the shorter lengths are very generally used. There are also cases where lengths of 1 000 ft. are essential. The results of tests with different lengths of pipe will be found in the tabulations of tests.

It has apparently been a matter of surprise and disappointment to some that no figures have been given as to the amount of material moved in dredging, and as to the cost of doing the work, based on cost per yard.

Where there is so much material moving from natural causes, it is impossible to arrive at anything like an accurate measurement of the amount of material that actually passes through the pumps when dredging in the channel.

Objection has also been raised to the method used in determining the capacity of the dredges, as the time involved in the barge test was so brief. At the suggestion of the writer, tests have been made recently which meet the last objection. They were made by the regular crews which have been engaged in channel dredging, and as nearly under the normal working conditions as practicable.

In making these tests the dredges were operated in a "pocket" behind a sand bar where there was no current to bring in or carry away material, and where the sand was very nearly the same as channel sand. It will be seen that these tests cover ample time to give good working results.

Before beginning the tests the site to be dredged was cross-sectioned with great care, and this was also done after the test was completed. This gave the data from which the volume handled by the pump was readily computed. The results obtained are as follows:

Dredge *Gamma*. Test made from October 31st to November 3d, 1898: Mr. Ockerson.

Time of actual dredging	45½ hours.
Total length of cut made	4 566 ft.
Average depth of cuts	7.1 "
Average advance per hour	100.3 "
Cost per lineal foot of cut	$0.0898
Total volume moved	45 856 cu. yds.
Average volume moved per hour	1 008 " "
Cost per cubic yard	$0.0089
Average steam pressure	145.6 lbs.
Average number of revolutions of main pump per minute	150.0

Dredge *Delta*. Test made from October 30th to November 3d, 1898:

Time of actual dredging	27 hrs. 23 mins.
Total length of cut made	2 711 ft.
Average depth of cut	6.5 "
Average depth of suction	13.6 "
Average advance per hour	99.0 "
Cost per lineal foot of cut	$0.0926
Total volume moved	34 462 cu. yds.
Average volume moved per hour	1 255 " "
Cost per cubic yard	$0.0073
Average steam pressure	151.1 lbs.
Average number of revolutions of main pump per minute	140.9

In both of the above tests the material was delivered through 1 000 ft. of discharge pipe.

The cost given only includes the cost of actual dredging time. The *Delta* began the tests with a defective cutter shaft which caused considerable delay; an unusual amount of driftwood was encountered, and this also caused delay. Including all these delays, together with time used in placing plant, changing cuts, etc., the cost becomes $0.147 per lineal foot and $0.0116 per cubic yard. On cuts 3 and 4 the *Delta* was pulled ahead at the rate of 138.3 and 177.5 ft. per hour, respectively, or as fast as the material could be carried away by the discharge pipes. The capacity per hour, under these conditions, was found to be 2 550 cu. yds., which may be considered as very near the maximum under favorable working conditions.

Dredge *Zeta*. Tests made November 3d to 6th, 1898.

The chief object of this test was to ascertain the facility with which a dredge with water-jet agitators could be operated through a sand bar, a portion of which extended above water. Otherwise the conditions were essentially the same as those noted in connection with the other dredges, except that the suction extended into mud which clogged the opening and developed a high suction head with an abnormally low discharge.

Mr Ockerson. The dredge readily cut her way into a dry bar for a distance of 550 ft. and an average width of about 200 ft. The suction was lowered to a depth of 17 ft. below the water surface, and the bar extended about 4 ft. above the water, giving a working face of about 21 ft. Cuts 1 and 2 were made with 500 ft. of discharge pipe, and the other four cuts with 350 ft. of pipe.

The results may be summarized as follows:

Total time of actual dredging	62 hrs. 55 mins.
Total length of cut made	1 785 ft.
Average depth of cut after dredging	11 + ft.
Average depth of suction below top of bar	20.6 ft.
Average advance per hour	31.1 ft.
Average cost per lineal foot	$0 307
Total volume moved	35 287 cu. yds.
Average volume moved per hour	562 cu. yds.
Average cost per cubic yard	$0.015
Average steam pressure	147 lbs.
Average number of revolutions of main pump per minute	180

This test must not be confounded with tests made under normal conditions, and the volume moved per hour is no measure of the capacity of the dredge under such conditions, but it serves a good purpose in showing what the dredge will do under the extraordinary conditions described. The small volume moved per hour may be attributed to the presence of mud in considerable quantities. This doubtless served to clog the suction and check the proper flow of material to the pump.

These tests, covering, as they do, a comparatively long period of time, give results which may be accepted as a fair measure of the normal working capacity of the dredges *Gamma* and *Delta*. The capacities given would probably be exceeded somewhat when working in channel sand, as the material is easier to handle than the finer sand found in the upper bars.

In closing this discussion it might be well to state that the results attained during the low-water season of 1898 have been very encouraging for dredging as a means of temporarily improving the low-water navigation of the river. It is true that the stage of the river throughout has been above the normal low water. The oscillations in the stage have been frequent, and this condition is, in general, detrimental to dredged channels.

Five dredges have been at work at such times as the stage was low enough to enable the suctions to reach the bottom. As a result of this dredging it can be said that in every case where dredging was done the depth was increased from 1 to 5 ft. for a navigable width of 250 feet or more.

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No. 839.

RESERVOIR SYSTEM OF THE GREAT LAKES OF THE ST. LAWRENCE BASIN;

ITS RELATION TO THE PROBLEM OF IMPROVING THE NAVIGATION OF THESE BODIES OF WATER AND OF THEIR CONNECTING CHANNELS.

By HIRAM M. CHITTENDEN.*

WITH A MATHEMATICAL ANALYSIS OF THE INFLUENCE OF RESERVOIRS UPON STREAM FLOW.

By JAMES A. SEDDON†, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

WITH DISCUSSION.

The immense system of inland navigation, known as the Great Lakes of the St. Lawrence Basin, possesses many features in common with the navigation of the high seas. These bodies of water are of great magnitude. The vessels which sail upon them rival in size sea-going craft, and are built to withstand storms which approach in fury those of the ocean itself. Light-houses and fog signals define the position of dangerous coasts, and breakwaters are required to protect the open roadsteads. The lake ports are essentially maritime ports, and the connecting channels resemble maritime straits and canals

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† U. S. Assistant Engineer.

more than they do ordinary rivers. The commerce of the lakes has now reached a magnitude which makes a respectable comparison with the ocean commerce of various parts of the world. Several of the lake ports do a business exceeded by but few seaports, and the commerce which passes the Sault Ste. Marie Canal and the St. Clair and Detroit rivers surpasses in volume that of the world's greatest maritime canal. Similarity of conditions has given rise to similarity of problems, such as the protection of ports, the deepening of harbors, and the enlargement of connecting channels, which are common objects in the interests of navigation, alike on the lakes and on the ocean.

There are, however, two important physical characteristics which create a notable difference between the conditions of ocean and lake navigation. One of these is the existence of diurnal tides on the ocean which presents problems of a distinct character not met with except in tidal waters. The other belongs to the lakes, and relates to their condition as bodies of fresh water with a perennial overflow. This characteristic not only gives rise to problems new and distinct in themselves, but it affords new solutions for old problems pertaining to maritime navigation in general. It is to a consideration of this particular feature of the navigation of the Great Lakes that the present paper is devoted.

Notwithstanding the vast magnitude of these lakes and their resemblance in many particulars to tidal waters, they are, after all, only parts of an immense river system which drains a large area of country. They are subject to the variable conditions of water supply characteristic of all streams. The connecting channels and the final outlet carry a continuous current, always in one direction, unlike maritime straits and canals through which the water may flow in either direction depending upon the state of the tide or of the wind. But while the lakes and their connecting channels thus constitute a great river system, that system differs from nearly all others in this particular—that the flow in the outlets is practically exempt from those variations and irregularities which obtain upon nearly all other streams.

The explanation of this most important characteristic is to be found, of course, in the controlling action of these lakes as great reservoirs interposed in the course of the stream. The storage represented by a few inches rise and fall in the lake levels is enormous, and when withheld during the flood season and released in the dry season, it gives to the outlets a regimen of flow which is radically dif-

ferent from that of an ordinary river. This influence is not annual merely, but cyclic as well, and a series of what may be called wet years causes a general rise of mean level, the storage of which maintains the flow in the outlets during the ensuing years of less than average precipitation. The balance of forces which Nature has here produced in the course of long ages is one of the most marvelous features of these lakes; and a careful contemplation of it cannot fail to convince one that an almost perfect compromise has been reached between the conflicting oscillations of lake level and outlet discharge. It is difficult to see how either could be brought nearer to absolute uniformity without a resulting departure in the other which would more than offset the gain.

Perfect as this natural condition is, however, it yet does not satisfy the ambition of those who make commercial use of it. There are many who consider the oscillation in lake levels as an evil which ought not to be suffered to continue. The ship-owner, who loads his boat to the full limit of harbor depth, complains when the waters subside and compel him to load to a lighter draft. The periodic oscillation in particular, which every now and then results in a continuous subsidence of mean level through a series of years, is very damaging to commercial interests and naturally causes a good deal of apprehension. Lake carriers are led, on account of it, to clamor for a corrective, which, if applied, might entail greater evils than those of a falling lake level, for they apparently forget that it is this very subsidence of level which keeps the water in the connecting channels from subsiding in a much greater degree.

Various causes have contributed to give this subject unusual prominence in recent years. A period of what might be called dry years culminated in 1895, in the lowest mean level in Lakes Michigan, Huron and Erie that has ever been experienced since the commerce of the lakes has been a matter of great moment. The artificial enlargement of the natural cross-sections of some of the connecting channels, for the improvement of navigation, has been thought by many to have contributed to this result. Simultaneously with the continuance of this unfavorable condition were the commencement and vigorous prosecution of a project which contemplates the permanent withdrawal of a large amount of water from Lake Michigan, the possible maximum of the diversion being 10 000 cu. ft. per second. All of these coincident

causes of alarm created no little uneasiness among commercial interests, and gave rise to a great deal of earnest consideration as to ways and means of averting the threatened dangers. Several public discussions and a considerable amount of professional literature on the subject were the first result. The matter finally came up in Congress in the shape of several reports which were made the basis of a bill* to provide for a thorough investigation of the whole question. In the meanwhile Nature commenced removing the immediate cause of apprehension, for the mean lake levels have been rising since 1895, and very likely will continue to rise for some time to come. As a natural result, the urgent clamor for action, which was so prominent a few years ago, subsided as the lake levels rose, and will probably not assume troublesome proportions again until a new period of low water arrives. The problem, nevertheless, is before us for solution, and the respite afforded by the natural relief now being enjoyed should not be permitted to go unimproved, but should be utilized to the utmost to bring about a well-digested plan of action before the hour of necessity again arrives.

This problem, as developed by the general discussions already referred to, aims to secure three objects:

(1) A control of the lake levels by which their annual and periodic oscillations may be, if not altogether eliminated, still materially reduced in range. This purpose is expressed in the following extract from the bill proposed to Congress in the winter of 1895–6, which directed, among other things, an inquiry and an investigation as to whether:

"it is practicable to control the waters of the Great Lakes and maintain them at substantially a uniform level at all seasons of the year, by a dam or dams or other works placed in Niagara River at the outlet of Lake Erie, and by a system of wing dams or other structures placed in the Detroit River and the St. Clair River, and the Sault Ste. Marie or St. Mary's River, at or near the respective outlets of Lakes Saint Clair, Huron and Superior."

The words "substantially a uniform level" are not specifically interpreted, but a limit of oscillation of 6 ins. has been urged as a practicable one, at least for Lake Erie.

(2) A permanent elevation of the lake levels by means of contracting works in the outlets. This, it is urged, would be the sim-

* This particular bill did not become a law.

plest, most effective and most economical method of securing a permanent increase of depth in the ports and connecting channels of the lakes.

(3) Some means of counteracting the tendency to a permanent lowering of lake levels due to an artifical enlargement of natural cross-sections of the connecting channels, or to the permanent diversion of any portion of the overflow to another water-shed.

It may be accepted as a preliminary condition, to which there can be no qualification, that any alteration of natural conditions in the Great Lakes, which shall result in a material diminution of flow in the outlets during any portion of the navigation season, cannot be considered for a moment. It therefore becomes a matter of the first importance to determine what effect the attainment of any of the above objects will have on the flow in these outlets.

Inasmuch as the navigation system of the Great Lakes, as already pointed out, is virtually a great river regulated by immense reservoirs distributed along its course, it is manifest that the hydraulic features of the problem are inseparably bound up with the laws of reservoir action in the regulation of stream flow.

Fundamental Propositions Pertaining to the Action of Reservoirs in the Regulation of Stream-Flow.

I.—*The effect of a natural reservoir upon a stream passing through it is to reduce variations and irregularities of flow and to give the stream a more uniform discharge upon leaving the reservoir than it had before entering.*

The explanation of this common phenomenon is obvious. When the inflow into a reservoir increases, the outflow will not generally increase at the same rate, for a portion of the inflow will be absorbed in storage due to a rise in the reservoir level. In like manner, when the inflow is decreasing, the outflow will not generally decrease at the same rate, for it will be reinforced by the storage which flows out as the reservoir falls. So that, while the total outflow may equal the total inflow, it will not generally show the same extremes of variation. The effect is analogous to that of a balance wheel upon the motion of machinery; the wheel being virtually a reservoir of force, in which irregular and sudden impulses are stored, and from which the stored force is paid out to sustain motion whenever there are sudden or irregular cessations in the application of the moving force.

To give mathematical expression to this proposition:

Let S represent supply in the most general sense of that term, positive and negative, including inflow from other reservoirs, rainfall, run-off, evaporation, and all other sources of supply or loss except the discharge of the outlet. It is expressed in second-feet and may have negative values.

Let Q represent the discharge of the outlet in second-feet.

Let h represent the variable height in feet of the reservoir surface above any datum, and also of the outlet immediately at the reservoir.

Let t represent the time in seconds from an assumed origin to any value of h.

Let A represent the area of the reservoir in square feet.

Then, at any instant, the rate of change in h will be represented by the following equation:

$$(S - Q)\, dt = A\, dh \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The equation for the discharge of the outlet is

$$Q = f(h) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which $f(h)$ always increases and diminishes with h, its particular form, in any given case, depending upon the character of the outlet.

From equation (1) it is seen that whenever h is increasing, dh is positive, and $Q < S$; and that whenever it is diminishing dh is negative and $Q > S$; also, that when h and Q are at maximum or minimum, dh is zero and $Q = S$.

In any cycle of changes, the maximum value of S precedes and is greater than the maximum value of Q. For if the curves of S and Q were both plotted to the same time scale, and if both maxima were equal and simultaneous, the two curves would be tangent to each other at that point where $Q = S$. But it has been seen that when Q is approaching its maximum it is less than S, and that after leaving its maximum it is greater than S. The two curves thus intersect at the point where $Q = S$, and therefore cannot be tangent; and since, at this point, Q is at a maximum and S is falling, S must have already passed a maximum greater than that of Q. In like manner, it will be seen that the minimum of Q follows, and is greater (algebraically) than that of S. The limits of oscillation in Q, therefore, lie within those of S.

From equation (1) it will also be seen that, other things being equal, the moderating effect of a reservoir upon stream flow varies directly with its area.

In the general proposition above stated the word "natural" was used by way of qualification, because in the practical uses to which artificial reservoirs are often put, the proposition is not strictly true. In some reservoirs the outlets are entirely closed during certain periods, and the water is all released at other periods, so that, in such cases, the natural fluctuations in the flow of the stream may actually be increased, or at least be radically altered in their periods.

Even in natural reservoirs there are apparent, if not real, exceptions. If a reservoir be very small in comparison to the volume of water flowing through it, but little larger in fact than the channel of the stream itself, the rise in the reservoir will follow so closely upon that in the stream that the two may appear to be simultaneous and equal. This is but one illustration of what may be stated as a general rule, that every stream is virtually a series of small reservoirs and connecting channels; and that all stream flow is more or less subject to the influence of reservoirs.

Reservoirs in Series.

As a corollary to the foregoing proposition, it follows that, if several reservoirs succeed each other in a descending series, so that a supply to any reservoir must flow successively through all below, the fluctuation of level in the different reservoirs, due to a variable supply to an upper reservoir, will diminish with a descent of the series and will approach zero as a limit.

It has been seen that in any reservoir the variation of Q is less than that of S, but, neglecting local supply, Q of one reservoir becomes S of that next below. Calling this S', and the discharge of the lower outlet, Q', there results:

$$Variation\ Q' < Variation\ S' < Variation\ S, \text{ etc.};$$

but $Variation\ h = \frac{Variation\ Q}{c}$ (see notation just below), in which c, as a rule in natural reservoirs, will increase with a descent of the series. The change in each term of the above ratio is therefore such as to diminish the value of *Variation h**.

* The term "variation," as used in this paper, is not to be taken in its mathematical signification, but simply as denoting the maximum fluctuation in S and Q. It corresponds to the term "oscillation," as applied to h.

There are possible exceptions to this rule. If it should happen that a particular reservoir were of very small area, with an engorged outlet, in which the value of c were actually smaller than that in the outlet next above, the value of $\frac{Variation\ Q}{c}$ might be greater than for the next preceding reservoir. Examples of this are common in the flow of nearly all streams. When the channel of a river is engorged from any cause, the value of c is less at this point than either above or below. If the river valley above be considered as a very small reservoir, we shall have a true example of the above exception. It is known, as a matter of fact, that the rise of rivers in times of flood varies materially at different points of their course owing to this cause. But while this exception is true of such extreme cases as have been here cited, it would not be true of any ordinary system of reservoirs.

For a more specific study of these relations:

Let A denote the area of the reservoir in square miles.

Let h denote the height in feet of the surface of any reservoir above the mean level for the cycle. For the first reservoir considered, it will be written h_1; for the next reservoir the resulting oscillation will be h_2, and so on to h_n.

Let c denote the variation of Q in second-feet for a change of 1 ft. in h, $f(h)$ being assumed to be a right line.*

Let P denote the coefficient of reduction of S, or the ratio $\frac{Q\ (\text{maximum})}{S\ (\text{maximum})}$.

Let p denote the coefficient of reduction of h, or the ratio of h for one reservoir divided by h for that next above, or $\frac{h_n\ (\text{maximum})}{h_{n-1}\ (\text{maximum})}$.

Let R denote the interval which elapses between the maximum or minimum of S in any reservoir, and the resulting maximum or minimum of Q. R is expressed in degrees of arc; a full circle, or 360°, representing a complete cycle, taken in this case as one year.

Let the variations of supply be represented by the equation $S = a \sin t$, in which a is the extreme variation in second-feet above or below the mean, and t is degrees of arc, of which 360 make a complete cycle.

* In streams of considerable magnitude and small variation in discharge, the assumption that $f(h)$ is a right line for slight changes in h, involves so small an error as not to impair in the least degree the validity of the mathematical processes in which $f(h)$ enters.

Then the following equations may be written:*

$$\text{Sin } R_n = \frac{1}{\sqrt{1 + \left(\frac{0.18c_n}{A_n}\right)^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

$$P_n = \frac{1}{\sqrt{\left(\frac{A_n}{0.18c_n}\right)^2 + 1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

$$p_n = \frac{0.18c_{n-1}}{\sqrt{A^2{}_n + (0.18c_n)^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

From these equations it is seen that, for a given cycle, R is a function of area and outlet alone, and independent of S; P also is a function of area and outlet alone; and p is a function of area and of inlet and outlet.

It will also be seen that R decreases as A increases, or as c of the outlet decreases; P increases as A decreases, or as c of the outlet increases; while p increases as A and c of the outlet decrease, and as c of the inlet increases. In each of the above cases the converse statement is true.

Inasmuch as the conditions which control the oscillation of level in natural reservoirs are seldom, if ever, such as to bring the crests of transmitted waves from upper reservoirs, and those due to local supply, simultaneously together, wave interference, which to some extent neutralizes the combined effect, is the result. In other words, the maximum oscillation of level in any reservoir of a series is never the sum of the maxima of local and transmitted oscillations, for these are never exactly superimposed; and they may even so occur that the hollow of the transmitted wave comes upon the crest of the local wave, and may actually cause a smaller oscillation than if there were no transmitted wave.

As a general proposition it may be said that, in a series of natural reservoirs, such as has been described, each with its independent local supply, the oscillation of levels will be mainly controlled by the transmitted supply when the reservoir area is relatively small, and by the local supply when its area is relatively large.

For the better illustration of the principles above enunciated, let the following example be considered. Let there be five reservoirs,

* See mathematical analysis hereto appended.

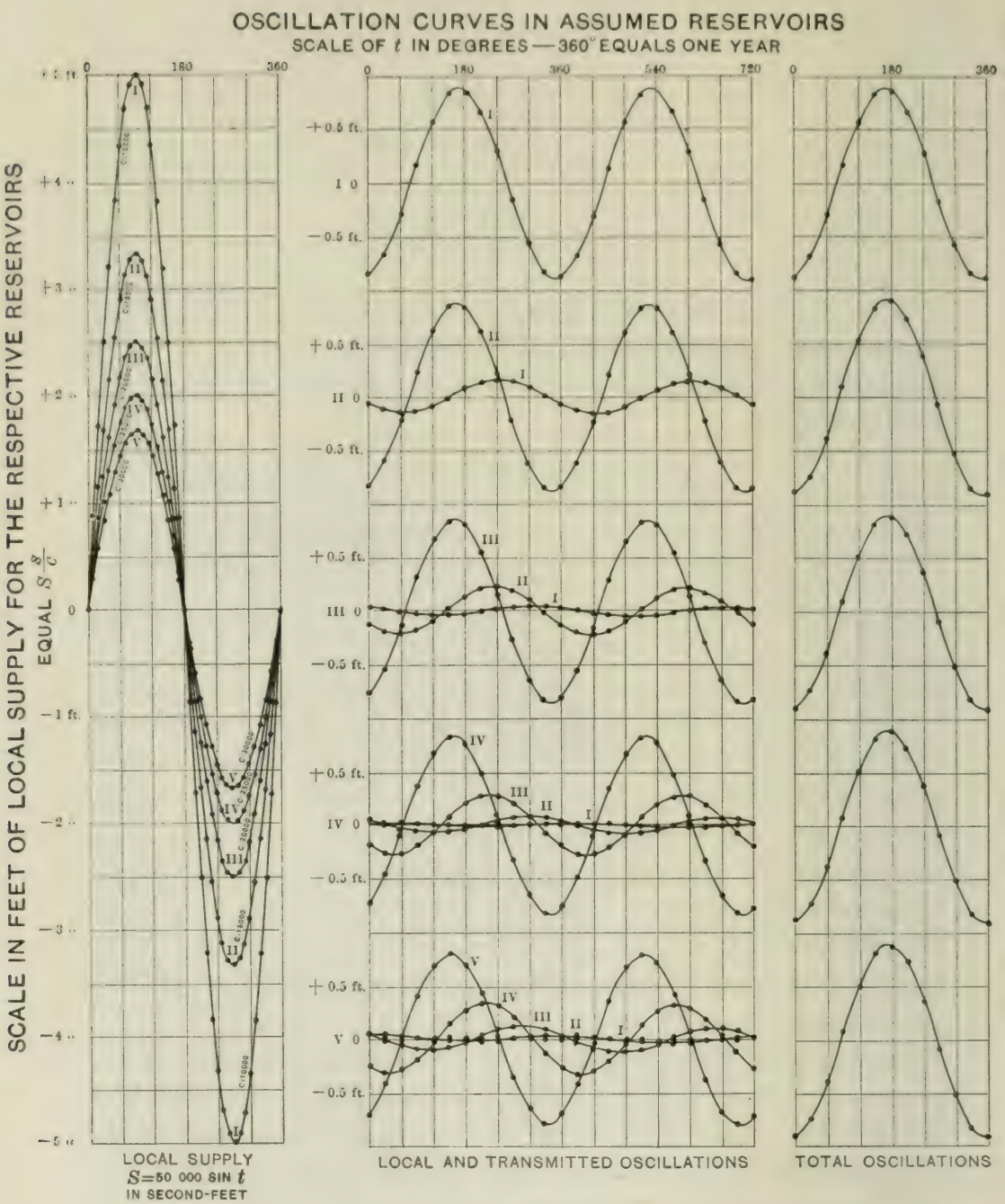

OSCILLATION CURVES IN ASSUMED RESERVOIRS
SCALE OF t IN DEGREES — 360° EQUALS ONE YEAR
SCALE IN FEET OF LOCAL SUPPLY FOR THE RESPECTIVE RESERVOIRS
EQUAL $S\frac{s}{C}$
LOCAL SUPPLY
S=50 000 SIN t
IN SECOND-FEET
LOCAL AND TRANSMITTED OSCILLATIONS
TOTAL OSCILLATIONS
+0.5 ft.
−0.5 ft.
I 0
II 0
III 0
IV 0
V 0

FIG. 1.

and assume the area of each to be 10 000 square miles, the range of variation in S to be 100 000 second-feet, and to be the same for each of the reservoirs; and the values of c, beginning with the first reservoir, to be 10 000, 15 000, 20 000, 25 000 and 30 000 second-feet. The values of R, P and p, as computed from equations (3), (4) and (5), may then be written as follows:

Reservoir.	R.	P.	p.
No. 1	81 days.	0.177	.....
No. 2	76 "	0.261	0.174
No. 3	71 "	0.339	0.254
No. 4	67 "	0.410	0.328
No. 5	62 "	0.475	0.396

The accompanying oscillation curves (Fig. 1) show for each reservoir the element of total oscillations which comes from local supply, and that from each of the reservoirs above, and enables one to judge at once by the eye what proportion of the total oscillation in any case is due to local supply, and what to transmitted supply. In the example above given, in which the reservoirs are relatively large, the oscillation is mainly due to local supply. For instance, in the third reservoir, the maximum oscillation, due to local supply, is 95½% as great as the actual oscillation, and occurs 12 days earlier. The maximum transmitted oscillation is 25½% as great as the actual, and occurs 68 days later.

To show what would be the effect upon a comparatively small reservoir, suppose that in the above series, in place of the third unit, there is introduced a reservoir of the following size and characteristics: Area, 400 square miles; range of variation in S, 4 000 second-feet; c of outlet, 20 000 second-feet. It will then be found that the maximum of oscillation, due to local supply, is 30% as great as the actual, and occurs 67 days earlier; while the maximum of oscillation, due to transmitted supply, is 94% as great as the actual, and occurs 20 days later. In this reservoir, therefore, the oscillation of level would be mainly controlled by the supply received from above.

II.—*Any reduction of the normal oscillation of level in a natural reservoir will increase the variation of flow in the outlet; and any reduction of the normal variation of flow in the outlet will increase the oscillation of level in the reservoir.*

From equation (1) it will be seen that, if the oscillation of level of a reservoir is to be diminished, $(S - Q)\,dt$ must be diminished, and consequently the values of $(S - Q)$ must be diminished; but since the variation of S from its mean is greater than that of Q, and since the variation of S is here assumed to remain unchanged, Q must be brought nearer to S, and its own variation must be increased.

The truth of the above proposition is readily apparent without mathematical demonstration. If a reservoir be prevented from rising by any amount, the water represented by the storage under normal conditions must be run out, and the flow in the outlet must be correspondingly increased. With a falling reservoir exactly the opposite process applies, and the flow of the outlet must be diminished by any restriction of the normal amount of fall. Finally, if the oscillation of level be eliminated altogether, Q will become equal to S—the condition of an unreservoired stream.

The importance of this proposition arises from its relation to the problem of controlling the oscillation of levels in the Great Lakes.

The principle laid down in Proposition II is at first sight inconsistent with equation (2). In discussing that equation it was seen that $f(h)$ increases or diminishes with h. But it has just been shown that a reduction of the fluctuations of h from its normal condition will increase the variation of Q, or $f(h)$. The explanation of this apparent paradox is, that any change in the normal variation of h[1] can only be brought about by some modification of the form or coefficients of $f(h)$; and that while h and $f(h)$ increase and decrease together for a fixed condition of the outlet, the rule has no application to a change from one condition of the outlet to another.

The converse of Proposition II is not necessarily true; that is, that an increase of oscillation in reservoir level, or of variation in outlet discharge, will diminish the variation or oscillation in the other. See remarks on artificial reservoirs under Proposition I.

III.—*In a series of reservoirs, such as has been described, if a permanent negative supply be introduced at any point, that is, if a portion of the supply be permanently withdrawn from any reservoir, it will cause a lowering of the mean level throughout the entire system below.*

This is apparent from equation (2) in which Q and h increase or diminish together. If, therefore, Q be permanently diminished, h must also be diminished, and the mean level of the reservoir will be lowered.

The lowering of the mean level in any reservoir, under the above assumption, will be such as to make $\frac{q}{c} = h'$, in which q is the negative supply and h' is the resulting fall in the mean level.

A permanent negative supply, however, unless of great relative magnitude, would have but little influence on the range of oscillation in the reservoir, provided the range of variation of S were not changed. If $f(h)$ were a right line, there would be no change in the oscillation, but if the function were of a form in which h appears at a higher power than unity, the oscillation of level would be increased by any lowering of the mean level.*

Under the assumption upon which Proposition III is based, the fall in mean level, resulting from the introduction of a negative supply, cannot be prevented by artificially increased storage in any part of the system. For assuming that $\int Q\,dt$ for the cycle is not to be changed, $\int + q\,dt$ must equal $\int - q'\,dt$, in which q and q' are the rates at which storage is accumulated during part of the cycle, and expended during the remainder. The total supply to the reservoir below is therefore not changed, and any increase during one period can be secured only by a corresponding decrease during a previous period. This might alter somewhat the oscillation of level, but would be powerless to prevent a lowering of mean level due to a permanent reduction of supply.†

Such a result can be prevented only by changing the form or coefficients of $f(h)$ in all outlets below the point where the negative supply is introduced—in other words, by contracting the outlets so as to diminish the normal discharge at mean level by an amount equal to the negative supply.

* This may be a proper place to notice a fact which is of some interest as a theoretical refinement, although of no practical importance.

The mean level of a reservoir will rise as its fluctuations of level are diminished, and fall as the fluctuations are increased—the mean discharge remaining the same—and the mean level will stand highest when there is no fluctuation of level, or when the flow in the outlet is constant.

This law results from the nature of $f(h)$ in equation (2). If it were a right line, that is, if the variation in discharge per unit variation in h were the same at all points of h, then the mean level would always be the same for the same discharge. But $f(h)$ is nearly always a curve in which its value increases more rapidly than that of h. In the weir formulas, for example, h appears at the $\frac{3}{2}$ power, while in ordinary streams the parabolic function, ch^2, is true within narrow limits. It is obvious, therefore, that $\int f(h)\,dt$ for values of h above the mean level, must be greater than for the same values below, and since the mean discharge during the cycle cannot be increased, the plane of mean level must be somewhat below that corresponding to a constant discharge. In general, however, the difference of mean level would be inappreciable.

† Referring to the last preceding note, it will be seen that if the storage in the reservoir be so manipulated as to reduce the oscillation of levels in the reservoirs below, the mean level would be slightly raised; and if so manipulated as to increase the oscillations, the mean level would be slightly lowered. But such effects would ordinarily be infinitessimal in value.

Name of Reservoir	Area of Water Surface sq. miles	Area of Water Shed sq. miles
Lake Superior	31,800	48,600
Lake Michigan-Huron	45,600	97,800
Lake St. Clair	495	5,100
Lake Erie	10,000	25,700
Lake Ontario	7,450	25,530

IV.—*Assuming the supply to remain unchanged, no modification of the normal oscillation of level in a reservoir, or of the normal variation of flow in the outlet, is possible except by some modification of the form or coefficients of* $f(h)$.

This proposition embodies the entire question of the character of works in the outlets of reservoirs designed to control or modify the normal oscillations of h and Q. Without entering at all into details of construction, it may be broadly stated that works of this class must take one of two forms, depending upon the immediate object to be attained. If that object be a reduction of oscillation of levels in the reservoir, then the controlling works must be of such a character as to increase the value of c [equations (3), (4), (5)]. If, on the other hand, the object be to restrict the variations of flow in the outlet, then the controlling works must be of such a character as to diminish the value of c. The first purpose may be accomplished by widening the outlet and diminishing the depth, as by the use of a long-crested weir around the head of the outlet, and the second by narrowing the outlet and increasing the depth.

Inasmuch as works of improvement, such as the deepening of natural channels, and works of simple contraction, such as wing dams, piers, etc., generally increase the depth and diminish the width, such works tend to uniformity in outlet discharge at the expense of uniformity of level in the reservoir.

Application of the Foregoing Propositions to the Reservoir System of the Great Lakes.

This system embraces five distinct reservoirs, the names and areas of which, together with the areas of the tributary water-sheds, are as follows:

Name.	Area of water surface.	Area of water-shed.
Lake Superior	31 800 square miles.	48 600 square miles.
Lake Michigan-Huron*	45 600 " "	97 800 " "
Lake St. Clair†	495 " "	5 100 " "
Lake Erie	10 000 " "	25 700 " "
Lake Ontario	7 450 " "	25 530 " "

* In this discussion Lakes Michigan and Huron will be considered as a single body of water.

† The areas in this table are taken from the report of the U. S. Deep Waterways Commission for 1896, except that the water-shed of Lake St. Clair, as given in that report, has been diminished by 1 220 square miles, and this area added to that of the Lake Erie water-shed. The water-shed map of the Commission report includes a considerable area in the St. Clair water-shed which would seem to drain into the Detroit River below a point where it could have any effect upon the levels of Lake St Clair.

The connecting channels are of considerable length and irregularity, but the retardation of effects, due to the time required for the water to flow through them, is probably in no case greater than twenty-four hours. It may be much less, if those portions are omitted which are practically inlets of the lakes, taking their stage directly from that of

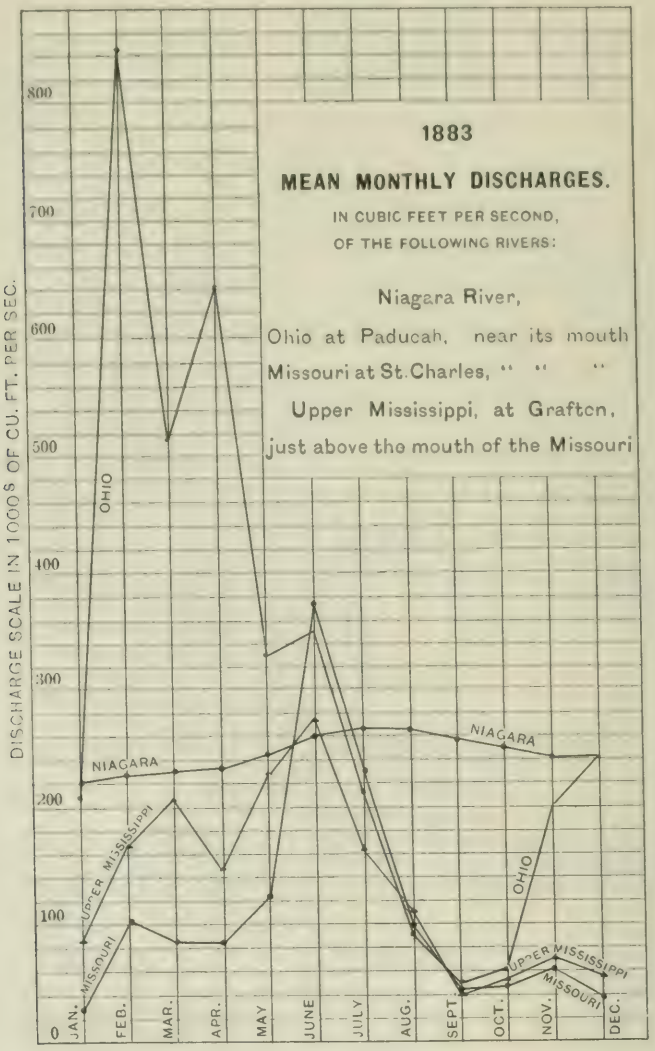


FIG. 3.

the lakes, and only those stretches are considered where the stage is dependent solely on the discharge.

Although existing data do not permit the precise determination of the values of c for the various outlets, they may be stated within a probable limit of error that will answer all the purposes of a rigid demonstration.

The values of c, here assumed, are as follows:

For Sault Ste. Marie River.....17 000 second-feet.
" St. Clair and Detroit Rivers.....26 000 " "
" Niagara River.....30 000 " "
" St. Lawrence River..... 34 000 " "

Deductions from Proposition I.

To appreciate fully the moderating effect of these reservoirs upon the stream flowing through them, as explained under Proposition I (page 359), it will be of interest to make a comparison with other well-known streams of similar magnitude. Take, for this purpose, the Ohio River at its mouth, the Missouri at its mouth, and the Mississippi at the mouth of the Missouri, and let them be compared with the Niagara at Buffalo. The areas of water-sheds of these four streams, and their mean discharge in second-feet, are as follows:*

	Niagara.	Ohio.	Missouri.	Mississippi.
Water-shed in square miles.....	265 095	205 750	530 810	171 570
Discharge in second-feet.....	232 800	307 000	100 000	130 000

The accompanying diagram, Fig. 3, exhibits graphically the variations in mean monthly discharge of these streams for the year 1883. The actual maxima and minima show, of course, a considerably larger divergence than the monthly means, the ratios $\frac{\text{(maximum discharge)}}{\text{(minimum discharge)}}$ for the above year being, for the Ohio, 28.22; for the Missouri, 29.00, and for the Mississippi, 10.29. For Niagara the ratio, disregarding wind effects, probably does not exceed, in any one year, 1.50.

In terms of absolute quantities, the storage by which this result is produced may be readily determined. The mean annual oscillation of level in Lake Superior, based upon 25 years' observations, is 0.93 ft., in Lake Michigan-Huron, 1 ft., and in Lake Erie, 1.16 ft. These oscillations represent an aggregate storage of 2 419 billion cubic feet of water—greater than the over-flow excess of the Mississippi River at Cairo in the great flood of 1897, and equivalent to about 153 000 second-feet for a period of six months. The maximum annual oscillation is about twice the above mean, and, of course, represents twice the amount of storage.

* The above data for discharge are based upon 25 years' record (1871–1895) for Niagara, and 6 years' record (1880–1885) for the other streams.

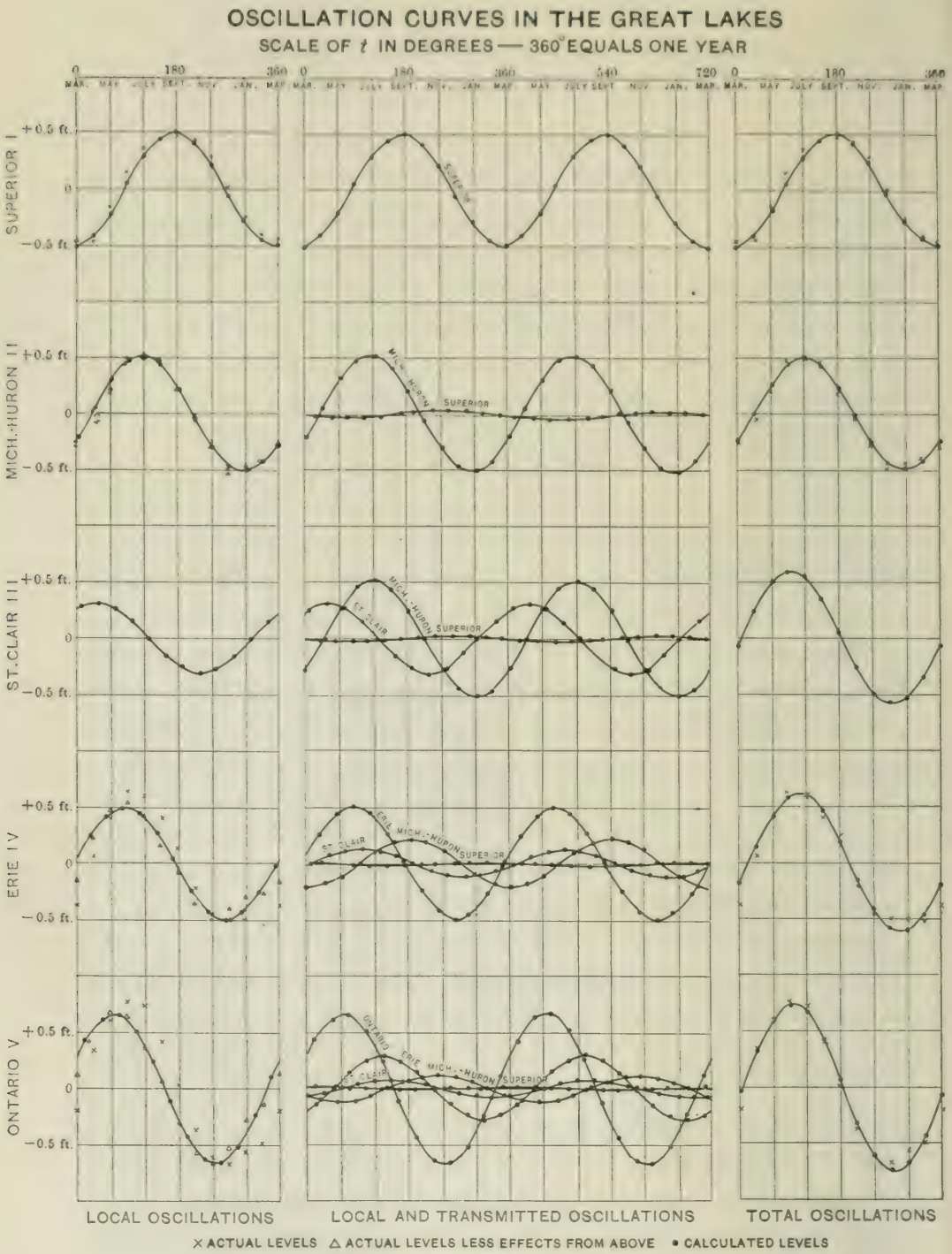

OSCILLATION CURVES IN THE GREAT LAKES
SCALE OF t IN DEGREES — 360° EQUALS ONE YEAR
SUPERIOR I
MICH.-HURON II
ST. CLAIR III
ERIE IV
ONTARIO V
+0.5 ft.
0
−0.5 ft.
LOCAL OSCILLATIONS
LOCAL AND TRANSMITTED OSCILLATIONS
TOTAL OSCILLATIONS
× ACTUAL LEVELS △ ACTUAL LEVELS LESS EFFECTS FROM ABOVE • CALCULATED LEVELS

FIG. 4.

Not less in magnitude and importance is the cyclic oscillation referred to earlier in this paper. During the period from 1872 to 1876, for example, the mean annual levels of the lakes above Niagara rose continuously, the aggregate storage being about 4 000 billion cubic feet. During the next three years the levels fell continuously, the aggregate loss of storage amounting to about 3 600 billion cubic feet.

The enormous moderating influence of these vast storage reservoirs thus practically eliminates, not only the regularly recurring changes in supply, but all sudden, erratic, or transient irregularities, and produces a flow in the outlets which, as already stated, is practically uniform from one year's end to the other.

In applying the formulas (3), (4) and (5), to the Great Lakes, no attempt has been made to go farther than to adapt them to the mean curves based upon 25 years' observations. To these, the ordinary sine curve, used in the analysis, has been fitted as closely as possible. The correspondence of the curves is not exact, but is near enough not to affect the general conclusions drawn from the discussion.

The following table gives the value of the retardation R, and the coefficients of reduction P and p, for the five lakes:

Name of lake.	R.	P.	p.
Superior	85.7	0.096	.....
Michigan-Huron	85.3	0.102	0.067
St. Clair	6.1	0.994	0.994
Erie	62.5	0.475	0.412
Ontario	51.3	0.635	0.560

This table, and the accompanying oscillation curves, Fig. 4, form a complete exhibit of the local and transmitted elements of the aggregate oscillation in each unit of the system.*

Column (2) explains the well known fact that the highest and lowest stages in the lakes occur long after the periods of maximum and minimum of supply have passed. However, from the curves and from column (2) the approximate mean dates of maximum or minimum supply for each of the lakes can be determined. To do this, take from the curve of oscillation due to local supply in any lake the dates of its maximum and minimum, and from these subtract the corresponding retardations in column (2). The results deduced by this process are

* The effect of the variable supply to Superior consumes 291 days in reaching Ontario, and its maximum is then only 1% of the total maximum oscillation in the latter lake.

given in the table below, and fall well within the probable limits of actual occurrence.

Name of Lake.	Date of actual h max.	Date of local h max.	Date of actual h min.	Date of local h min.	Retardation.	Date of S max.	Date of S min.
Superior	Sept.	Sept.	Mar.	Mar.	86	June	Dec.
Michigan-Huron	July	July	Dec.	Jan.	85	April	Oct.
St. Clair	July	April	Dec.	Oct.	6	April	Oct.
Erie	June	June	Feb.	Dec.	63	April	Oct.
Ontario	June	May	Dec.	Nov.	51	April	Sept.

The quantity S, it must be borne in mind, is a very complex one. It is the result of at least three important and ever active causes—precipitation upon, and evaporation from, the surface of the reservoir, and run-off from the tributary water-shed. The first and third elements are always positive and the second always negative; but their maxima and minima are rarely coincident, and probably none of them coincide very closely with the actual aggregate. Precipitation, as based upon many years' observations, has two yearly maxima, occurring usually in June and September, and two minima, occurring usually in August and February. Run-off has its maximum generally in those months when the melting of the winter snows occurs. This is ordinarily in March for the lower lake region, and in April for the Lake Superior region. The minimum of run-off spreads over a pretty long period, ranging from September to February, and is controlled, not only by actual precipitation, but by the influence of cold weather in congealing precipitation and checking the flow of streams. The maximum of evaporation occurs generally in July and August, and the minimum in the period from December to February; but there are certain phenomena, to be referred to later on, which indicate a higher winter rate of evaporation than is generally supposed.

The complex and variable elements, of which the actual local supply for any of the lakes is made up, render it extremely hazardous to predict from them the time of actual maximum and minimum; for these may incline toward any of the component elements which happen at the time to predominate. No method seems so rational as that above applied of working backward from the known dates of maximum and minimum oscillations due to local supply.

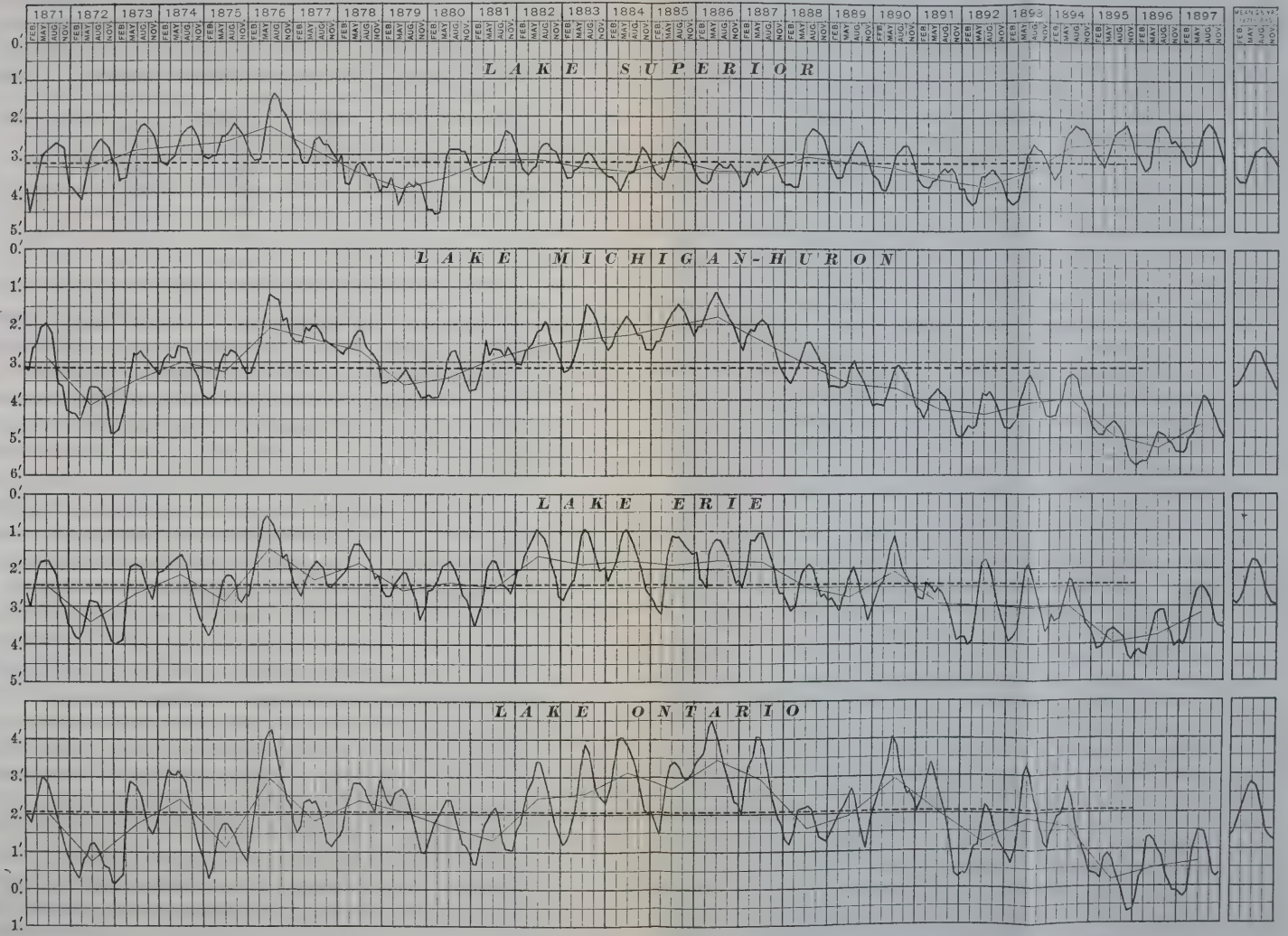
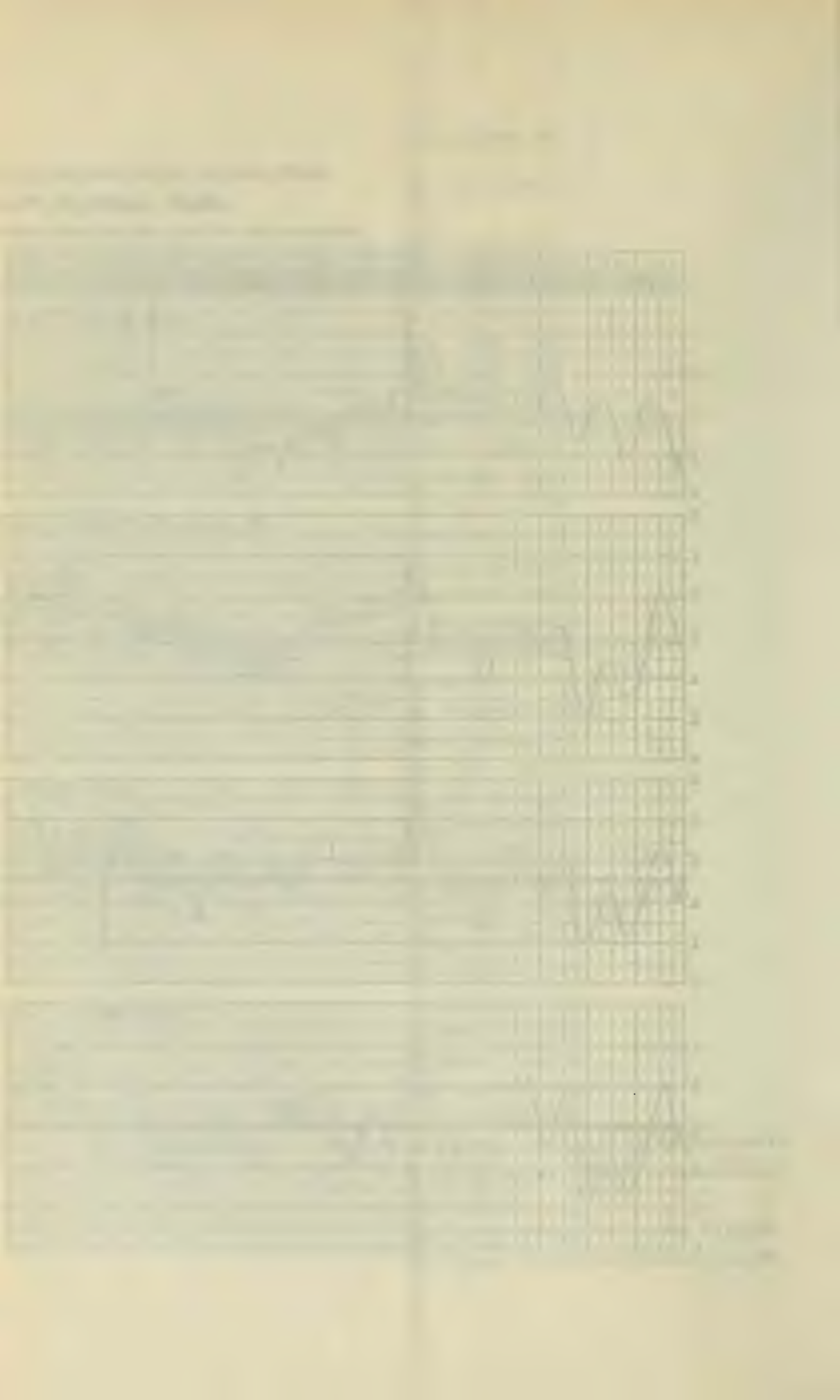
Assuming the mean discharge of the outlets to be, for Superior, 75 000 second-feet; for Michigan-Huron, 195 000 second-feet; for St.

PLATE XXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XL, No. 839.
CHITTENDEN AND SEDDON
ON LEVELS OF THE GREAT LAKES.

CHART SHOWING THE OSCILLATIONS OF LEVEL AND OF MEAN ANNUAL LEVEL OF THE

GREAT LAKES OF THE ST. LAWRENCE BASIN

FROM 1871 TO 1897 INCLUSIVE, WITH MEANS OF TWENTY-FIVE YEARS FROM 1871 TO 1895 INCLUSIVE.



Clair, 200 000 second-feet; for Erie, 225 000 second-feet; for Ontario, 260 000 second-feet; the corresponding local supply for each of the lakes in thousands of second-feet per month, based upon the twenty-five year curves, is found to be as follows:

Month.	Superior.	Michigan-Huron.	St. Clair.	Erie.	Ontario.
January	— 2	+111	+19	+43	+62
February	+ 31	+185	+21	+52	+74
March	+ 75	+235	+20	+55	+75
April	+115	+255	+15	+49	+66
May	+152	+240	+ 8	+37	+49
June	+164	+191	+ 1	+23	+27
July	+153	+129	— 5	+ 7	+ 8
August	+119	+ 55	— 7	— 2	— 4
September	+ 75	+ 5	— 6	— 5	— 5
October	+ 31	— 15	— 1	+ 1	— 4
November	— 2	+ 0	+ 6	+ 13	+21
December	— 14	+ 47	+13	+27	+43

The results here given are to be taken only as general approximations, in the absence of precise data as to the mean discharge of the outlets; but they serve to show the fact, intended to be brought out here, that at certain seasons there is a negative supply on all of the lakes. The cause will naturally be attributed to the preponderance of evaporation at such times over rainfall and run-off; but the perplexing feature of the phenomenon is that in one lake in particular—Superior—this negative supply comes in months when evaporation is supposed to be at a minimum. An examination of Table No. 2 will show that on all the lakes there are large deficiencies at times, even in winter months. No explanation suggests itself except that evaporation from ice and snow on the lakes may be greater than is commonly supposed.

Relative Effects of Local and Transmitted Supplies.—Plate XXXVI, showing oscillation curves, affords a striking illustration of the observation made in discussing Proposition I (page 359), viz., that, in the general case, the oscillation of levels will be controlled by the transmitted supply when the reservoir is relatively small, and by the local supply when it is relatively large. Thus, the maximum of oscillation in Michigan-Huron, if due to local supply alone, would be 4% greater than it actually is, and would occur about three days earlier. The maximum of oscillation, if due to the transmitted supply from Superior alone, would be only 7% as great as it actually is, and would occur 141 days later.

For St. Clair, the local maximum would be about 53% as great as the actual, and would occur about 57 days earlier; while the trans-

mitted maximum would be 83% as great as the actual, and would occur 32 days later.

For Erie, the local maximum would be about 80% as great as the actual, and would occur about 20 days earlier; while the transmitted maximum would be about 39% as great as the actual, and would occur 51 days later.

For Ontario, the local maximum would be about 88% as great as the actual, and would occur about 28 days earlier; while the transmitted maximum would be about 47% as great as the actual, and would occur about 64 days later.

From these figures, and from the curves, an excellent idea may be had of the parts which local and transmitted supplies play in the actual oscillations of the lakes. Except in St. Clair, it is seen that the local supply is the controlling factor, and that in one lake—Michigan-Huron—the transmitted supply actually diminishes the total fluctuation. In St. Clair the transmitted supply controls—as was to have been expected on account of its relatively small area,

It is also plain that any attempt to control the fluctuations of one lake by modifying the inflow from another will produce only insignificant results.

In like manner it is manifest that the oft-asserted mutual dependence of levels between Michigan-Huron and Erie, evidence of which, it is urged by some, is the fact that their difference of level scarcely ever varies as much as 1 ft.,* has no foundation in fact. St. Clair completely interrupts the continuity of flow on account of the fluctuation due to its own local supply.†

* Lake Michigan-Huron and Lake Erie rise and fall together, not because of any mutual dependence of levels, but because similar climatic forces are operating simultaneously in both basins. The fact that their levels do not move in closer unison—that sometimes Erie is falling when Michigan-Huron is rising, and *vice versa*—still further negatives the probability of any very intimate relation between the oscillations of level in these two bodies of water.

† The influence of St. Clair upon the discharge of the Detroit River may even be greater than has been here assumed. Unfortunately there are no actual data as to the oscillation of levels in this lake, and it has been necessary to deduce a curve from known data, such as the area of the lake, the inflow from Michigan-Huron, and an assumed S, based upon the similarity of climatic conditions on the watersheds of Michigan-Huron, Erie and St. Clair. In this, however, an error may have been made in assuming an eccentricity for the St. Clair curve of supply but little greater than that for the other lakes. The water-shed of St. Clair, compared with the lake area, is relatively much larger than those of the other lakes, as may be seen from the following ratios of lake to land area:

Superior	1.5
Michigan-Huron	2.1
St. Clair	10.3
Erie	2.4
Ontario	3.4

Remembering that S is the algebraic sum of precipitation on the lake surface, evaporation from it, and run-off from the water-shed, a little consideration of the character of these elements will show that as run-off predominates over rainfall and evaporation, the eccentricity of S will increase. It is quite possible, therefore, that the range of local supply for St. Clair has not been assumed large enough, and that the oscillations of level in that reservoir are due, to a greater extent than has been admitted, to local supply.

Deductions from Proposition II.

Table No. 2, appended to this paper, shows in detail, month by month for twenty-five years, and for each of the five lakes, the amount of storage accumulated or lost, both in terms of h and in thousands of second-feet per month. It also shows similar data for the mean of the twenty-five years. Table No. 3 shows the oscillation of mean annual level in feet in each of the lakes from 1871 to 1895, inclusive, with the equivalent storage in thousands of cubic feet for the period of one year. Table No. 4 shows the rate in second-feet at which storage corresponding to a rise or fall of 6 ins. on each of the lakes would have to be accumulated or expended in periods varying from one to three months. The tables show, not only the data for each lake by itself, but the cumulative effects in descending the series, supposing the causes to be operating simultaneously throughout the system.

These tables supply ample data for conclusions as to the practicability of controlling the levels of the lakes within fixed limits—say, 6 ins. Thus, let it be assumed that the outlet of any of the lakes, Superior, for instance, is to be so regulated that the annual oscillation of the lake level shall not exceed 0.5 ft. The amount of storage represented by a depth of 0.5 ft. on Lake Superior is equivalent to 168 000 second-feet for one month. Now, during the months from March to September, in 1871, for example, storage was accumulated in this lake at the following rates in thousands of second-feet: March, 146; April, 168; May, 178; June, 40; July, 24; August, 20; September, 34. Assuming that this storage is to be reduced to an equivalent of 168 000 second-feet for one month, and that the reduction is to be taken from the months of most rapid accumulation, it would give the following results: It would reduce the storage in March by 38, in April by 60, and in May by 70, and would leave for each of these months 108 000 second-feet* which would have to be run out in addition to the normal discharge. This, it will be understood, is based upon the impossible supposition that climatic conditions can be foreseen so as to apply the allowable storage of 0.5 ft. to those months of most rapid accumulation. In actual practice, of course, no such result could be realized, and the increase of outflow would be greater than above indicated.

Applying this process to the ensuing period of falling levels, from October, 1871, to April, 1872, it will be found that for the months of

* For values of mean discharge of the various lakes, see p. 375.

December and January the outflow would have had to be reduced by 94 000 second-feet *, and for the other months, during which the lake was falling, by the full amounts given in the table. It requires no comment to show, not only the impracticability, but the utter impossibility of any such accomplishment.

However, the difficulty here encountered would evidently be less in the lower units of the system, if considered by themselves, for there the reservoir area is much less and the flow of the outlets much greater. To see what effect would be produced upon the flow of Niagara by a reduction to 0.5 ft. of the oscillation in Erie, apply the above process to that lake for the year 1871–72. It will be found that the outflow would not have been increased by more than 24 000 second-feet, nor restricted by more than 32 000 second-feet. There are other years, however, in which the increase of outflow would have been much greater, but it may be assumed that any practicable restriction of the oscillation of Erie to a limit of 0.5 ft. would probably not often increase or decrease the flow in the outlet by more than 30 000 second-feet.* Whether a change of this amount in the normal variation of flow of Niagara, particularly in the low-water period, could be admitted, is at least an open question, and one that would have to be settled before regulation could actually take place.

If regulation to a limit of 6 ins. were simultaneously applied to all the lakes, or even to Michigan-Huron and Erie, the cumulative effect on Niagara would render the project wholly impossible.

It will probably impress any one who carefully examines this subject that an interference with the normal annual oscillation of levels of the lakes will at best be an uncertain and hazardous undertaking; and that, if such interference were to take place, it might better be for the purpose of increasing, rather than diminishing, the oscillations of levels in the reservoir, and of thus reducing the variations of flow in the outlet.

It is, after all, not so much the annual oscillation that gives rise to complaint on the part of navigators as the long periodic oscillation. The real source of trouble is the continuous subsidence of levels through several years, by which the carrying power of the great ships is constantly reduced, and their earning capacity diminished. Can these periodic oscillations be eliminated, and can the mean annual levels be always maintained above a minimum plane?

* For values of mean discharge of the various lakes, see p. 375.

So far as the rise is concerned, the problem is certainly feasible, for the increased flow, due to its elimination, when spread over such long periods, would not be objectionable, even in its cumulative effects.

The case is not equally clear in regard to the subsidence of levels. An extreme example of the fall of the mean annual levels of Michigan-Huron and Erie is that of the year 1894–95. Michigan-Huron dropped during that period at a rate of 42 000 second-feet, and Erie at the rate of 8 220 second-feet; but Superior in the meantime rose by an average of 1 700 second-feet. To have prevented the subsidence of levels in Michigan-Huron and Erie would have required a uniform reduction of the normal discharge of Michigan-Huron by 40 300 second-feet, and of Erie alone by 8 220 second-feet, while the cumulative effect in Erie would have been 48 520 second-feet. This being a period when the normal discharge of these rivers was injuriously small, these reductions would probably not have been admissible during the navigation season. If restricted to one-third of a year, that is, to the winter season, they would have been 120 900 second-feet for Michigan-Huron, 24 663 second-feet for Erie alone, or 145 560 second-feet as the cumulative effect from Michigan-Huron and Erie. As the normal discharge of Michigan-Huron at this time was probably between 150 000 and 175 000 second-feet, and of Erie not much more than 200 000 second-feet, the elimination of the cyclic fluctuation alone would, it is seen, have involved a reduction of discharge in the outlets, during the entire winter season of four months, of nearly 75 per cent.

Should full investigation show that this amount of reduction in the discharge of the outlets during the non-navigation season could be permitted without injury to any interests, the possibility of uniform regulation, such as is here assumed, would still be dependent upon a condition which can never be realized. It is impossible to forecast climatic conditions for anything like the periods covered by these cycles. Those charged with the control of the outlets must always be for the most part in the dark as to what is going to happen in regard to rainfall and run-off. It is therefore clearly impossible to fix in advance, even for the period of a single year, a uniform rate at which the discharge of the outlets must be increased or diminished in order to prevent a change in the mean annual level. The only way to accomplish this result that suggests itself would be to accumulate an amount of storage in the lakes during each winter, and the ensuing

period of high water, equal to the maximum subsidence which experience shows may be expected to follow during the low-water season. If this were done, the levels could always be maintained above a minimum plane. This result, however, could be attained only at the cost of increasing now and then the annual oscillation.

Of course, to make any regulation of the lake levels practicable the mean levels must be raised somewhat, in order to be able to secure the increase of outflow which would at times be necessary; but this elevation of mean level is in itself one of the ends contemplated by the general scheme of lake level regulation. Taken by itself it is unquestionably a practicable matter, at least within small limits. Such a step would indeed involve grave difficulties in the outlets themselves, to which reference will presently be made, and will always be complicated with the question of damages, but so far as the simple matter of raising the levels of the lakes is concerned, there can be not the slightest doubt of its feasibility. It is in fact claimed that this has already been accomplished in Lake Superior by works placed in the outlet of St. Mary's River for other purposes. These works consist of bridge piers and water-power works, and the contraction of the outlet has been such as to give an estimated rise of 0.5 ft. in the level of the lake, prior to January, 1896.*

The question of permanently raising the levels of the lakes is, or may be made, entirely independent of that lake level regulation. A simple contraction of the outlets will accomplish the first purpose, but not the second. It would in fact increase the oscillation somewhat, in accordance with a principle brought out in discussing Proposition IV (page 369), that if the outlet of a reservoir be narrowed or deepened, the value of c will be diminished, and the oscillation of level will be increased while the variation in flow will be lessened.

Deductions from Proposition III.

The third of these general propositions relates to the effects upon the lake levels, of the diversion of any portion of the supply of the

* Report by E. E. Haskell, U. S. Asst. Engineer, dated January 11th, 1896. Although it is true that the level of Lake Superior was rising for several years prior to 1896, while the levels of the lower lakes were falling, still the climatic conditions are dissimilar enough to lead one to hesitate in attributing this particular rise to artificial causes alone.

Great Lakes to other watersheds, and to the methods of counteracting such effect. The specific application of the proposition now in view, is to the Chicago Drainage Canal with its contemplated diversion of 10 000 second-feet, and to the scheme advocated by some engineers of diverting a much larger amount, say 30 000 second-feet, to help out the low-water navigation of the Mississippi. That such diversions will permanently lower the levels of all the lakes, below and including that from which they are taken, is certain. The amount of this lowering is a function of c of the various outlets, and its correct determination depends upon the prior correct determination of this coefficient.

Among the suggestions advanced for counteracting this lowering effect is that of storing water in Lake Superior to make up the amount of diversion. A reference to the discussion of Proposition III (page 366) will show the fallacy of this scheme. The storage of water in Lake Superior must commence by cutting off the outflow. When the stored water is run out, the total increase of flow over the normal condition will only be equal to the previous decrease. In other words, the total supply to the lakes below, upon which their mean level depends, cannot be altered a particle by storage.

The only way the effects of diversions can be counteracted is to contract the outlets, so that the flow through them at the normal mean level shall be diminished by an amount equal to the diversion.

Analogous to the effect which must result from the permanent withdrawal of any portion of the supply of the lake system to other water-sheds is that due to an increase of the cross-sections of the outlets resulting from improvements in the interests of navigation. Such works must necessarily result in a permanent lowering of mean level in the reservoir above. The remedy is, of course, to contract the outlets sufficiently to diminish the outflow at the normal mean level by an amount equal to the increase caused by the enlarge ment of section. In superficial area the contraction would have to be greater than the previous enlargement, for the whole change thus made in the cross-section is in the direction of deepening and narrowing the outlet; and a deep and narrow channel will carry more water than a wide and shallow one of the same area of cross-section.

Deductions from Proposition IV.

All works intended to change the levels of the lakes, to control their oscillations, to counteract diversions, or to regulate the flow of the outlets, must be placed directly in, or at the head of, the outlets themselves. The two broad classes into which such works must fall have already been pointed out, viz.:

(*1*) Works tending to increase the width and diminish the depth, with the result of diminishing oscillation of level and increasing variations of discharge; and

(*2*) Works tending to contract the width and increase the depth, with the result of increasing oscillation of levels and diminishing variations in discharge.

The only example of a work under the first class is that proposed by some engineers for controlling the level of Lake Erie. For obvious reasons, the regulation of the levels of this lake is of more importance than that of any of the others. There need only be mentioned its vast harbor interests, and the fact that at least two, and probably three, canals will in the future lead directly from its lower extremity to Ontario or to the Hudson River. A fixed depth in these harbors and over the miter sills is of great importance. To secure uniformity of level it has been proposed to enclose the head of Niagara by a fixed weir, which will give the desired permanent increase in elevation of level, and which shall be of such length that a change of depth on the crest of, say, 6 ins. will so modify the outflow as to prevent a rise or fall of level of more than that amount.

Two objections appear to be conclusive against this arrangement: (*a*) The very advantage which first commends it, that of automatic regulation of oscillations, would, in reality, prove a serious disadvantage; and (*b*) the whole result can be accomplished in a simpler, more effective, and less expensive way by works placed directly in the outlet.

The trouble with the automatic action of the long-crested weir is that it would always give the same discharge for the same depth of overflow; whereas, the best regulation of the discharge might require that it be not always the same for the same stage. It has been seen that a regulation of the levels of Erie within a limit of 6 ins. would increase or decrease the normal flow in Niagara at times by as much as 30 000

second-feet; but the range of the normal variation itself above or below the mean doubtless frequently amounts to 30 000 second-feet. The weir must therefore be of such length that a variation of 6 ins. in the depth of flow over its crest will give a variation of discharge of 60 000 second-feet. When it is considered that changes of 2 to 4 ft. in gauge readings due to wind effects "are of so frequent occurrence as to be considered common,"* and that changes of from 4 to 8 ft. are not rare, the effect of a long weir upon the discharge of Niagara, in times of heavy winds, would, it is readily seen, be disastrous. With a rising lake the matter would be of less consequence; for if the level of the lake were regulated but a foot or two above its present mean stage, the increased overflow would choke up the outlet and drown out the weir: but with a falling lake, no such counteracting effect would be experienced, and Niagara River would frequently run dry.

As before stated, the whole result expected from the long weir can be obtained in a simpler, more effective and less expensive way, by works placed directly in the outlet. The form of structure which appears to be essential to a rational regulation of the levels of the lakes is one that shall be in a measure under the control of human agencies. Disclaiming any purpose of proposing technical or specific details of construction, the following general plan may be suggested as embodying what would seem to be the most practicable method of control. At a suitable section of the outlet, preferably where rock foundation may be had, let a series of piers, similar to bridge piers, be erected at proper intervals, whose aggregate area of cross-section below the water surface will be equal to the contraction of the channel section, which is necessary to secure the desired permanent elevation of the mean level of the lake. These piers would, thus, by themselves, accomplish that part of the purpose of regulation which relates to a permanent elevation of level. For the purpose of controlling the oscillations, some arrangement by which the area of the new cross-section can be enlarged or diminished is necessary. To accomplish this, let a sill be anchored to the bottom of the river from pier to pier, and let a bridge be laid from pier to pier, but slightly elevated above the water surface. Upon this bridge, at convenient intervals, let proper arrangements be made for the handling of good-sized needles

* Report of the United States Deep Waterways Commission, for 1896, p. 156.

which may be lowered into the river and supported by the bridge at the upper end, and by the sill at the lower end. Or, better still, these needles could be fixed permanently in position, but balanced on the longitudinal axis, so that the broad flat surface or the narrow edge, as desired, could be presented to the current. These needles, when in place, need not be adjacent, but there may be open intervals between them; the main point being that their aggregate area shall be sufficient, by proper manipulation, to give the desired control over the flow in the outlet. A navigable pass of sufficient width should be left entirely free from obstruction. With an arrangement of this sort, all the advantages of the fixed weir may be realized, with the very important additional advantage of adaptation of control to actual conditions as they arise. Such a structure ought to be much cheaper than a long weir. Its care and management would cost something, but the expense from this cause ought not to be great.

One important drawback will always be encountered with any form of contraction that can be devised. The raising of the level of the lakes will increase the slope at the site of the works of contraction, and may develop currents of such rapidity as to interfere seriously with the passage of boats. At the Sault Ste. Marie this is a matter of less importance, for locks there are a necessity any way, and the only effect would be to increase the lift. Even this might not result if Michigan-Huron and Superior were each raised by a like amount. But in passing from Huron, St. Clair or Erie into the rivers below, the case is different, and contracting works might, and probably would, develop strong currents and sharp slopes in their immediate vicinity.* It is doubtful if the simultaneous and equal raising of Erie and Michigan-Huron would obviate the difficulty, for the increased channel depth would diminish the slopes, and would tend to concentrate the fall at the contracting works. It might be necessary to have a series of works, one below another, with the area of contraction gradually diminishing so as to let the slope down gently by distributing it over a considerable distance. In any event, the use of locks would be too serious an inconvenience to merit consideration.†

* The outlets of both Michigan-Huron and Erie have engorged sections where they leave the lakes; and even in their natural conditions show marked slopes and high velocity of flow at these points.

† It is scarcely necessary to invite attention to the international feature of this problem. No radical step in the direction of lake-level regulation can be taken, except with the co-operation, or at least consent, of Canada. It is, in fact, by no means improbable that this feature of the question may prove the most embarrassing of all.

Physical Data.

It will have been observed that, in this discussion, the primary data upon which the oscillation of lake levels depends—those, namely, of precipitation, evaporation, and run-off—have been dealt with scarcely at all. In fact S does not appear in equations (3), (4) and (5), but only the quantities A, h and c. The discussion of the reservoir question therefore depends upon a knowledge of the area of the lakes, the oscillation of the levels, and the characteristics of flow in the outlets. The first of these elements is known with precision. The second has been a matter of record for nearly forty years, and in this discussion is based upon a period of observation of twenty-five years. The data are not, indeed, all that could be wished. The records have not been made up from self-registering gauges, as would have been the better plan, but from observations taken at a particular hour each day. Considering the sudden effects of winds and barometric changes on the levels of the lakes, it is clear that the individual records cannot be of much value; but since erroneous values are liable to occur as much on one side of the truth as on the other, the monthly means of daily readings are probably not far from correct, while the mean of so long a period as twenty-five years must be very accurate.

Of the discharge of the outlets comparatively little is yet known. In this matter, however, it is not so much the absolute value of the mean discharge that is needed, as the variations from the mean. In other words, Q is of less importance than c. In the assumption of c it has been necessary to proceed somewhat in the dark, but nevertheless with confidence, because a considerable variation in c will not materially affect the results with reservoirs of such magnitude as the Great Lakes. In like manner the assumption that $f(h)$ is a right line is so near the truth that its departure therefrom cuts no figure whatever in the general result. It is confidently believed, therefore, that the general conclusions here arrived at will not be essentially modified by more exhaustive data.

It is, nevertheless, of great importance to a final solution of the problems of the lakes that these data be made as complete as possible. The future records of the lake levels should be kept with self-registering gauges, and there should be one of these for Lake St. Clair as well

as for the other lakes. The discharge measurements should embrace every phase of flow in the outlets, and particular care should be exercised to secure simultaneous measurements at various stations along the channels connecting Huron with Erie. It is a matter of congratulation to know that these important steps are now in contemplation by the existing Deep Waterways Commission.

Concerning S, the collection of data appears to be altogether of secondary importance, and this for the excellent reason, that, having A, h, c and Q, S can be determined from them more accurately than in any other way. If the difficulties in the way of a satisfactory determination of any one of the elements of S—precipitation, evaporation, or run-off—are considered, it will be conceded that an approach to the problem from that direction is wholly injudicious. Take Lake Superior, for example. The precipitation at any one of the points, Duluth, Marquette, Sault Ste. Marie, White River or Port Arthur, around the shore of the lake, varies materially from that at any other point. Admitting that correct records can be kept at these points, how is the true proportion of lake area to be assigned to each in order to determine the average precipitation over the whole?

The question of evaporation is still more unsatisfactory. There is no practicable method of determining it accurately. How are the effects of winds on such great bodies of water to be determined? It is known that waves increase the exposed area about $2\frac{1}{2}\%$, while the spray from breakers adds still more; but at the same time the commotion of the water brings up the cooler depths to the surface and lowers the temperature upon which the evaporation largely depends.*

Again, observations taken near shore might not hold for the interior of the lake, where the atmosphere may reasonably be assumed to be more laden with moisture and to be of a lower temperature. The element of evaporation from ice and snow would be a very uncertain one, owing to the absence of any definite knowledge of the extent of ice-covered areas. With these inherent uncertainties in the problem of determining the rate of evaporation on the lakes, great confidence can hardly attach to any attainable results.

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* In midsummer the surface temperature on Lake Superior was found by Geo. Y. Wisner, M. Am. Soc. C. E., to fall 5 or 6° after heavy winds.

The run-off from the land area of the basin could be determined with tolerable accuracy for any one year, by a continuous gauging of a sufficient number of streams, but the results would be of little value for another year in which conditions might be very different.

It is therefore not apparent why this particular line of data should receive special attention. The complex expression which gives their combined effect is already integrated in the rise and fall of the lake levels and in the flow of the outlets, and if these are known, S can be determined from them better than it can ever be measured.

TABLE No. 1—Water Levels of the Great Lakes.

[From the Report of the United States Deep Waterways Commission. 1896.]

Lake Superior, at Superior, Wis., and Marquette, Mich.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 604.76 ft.

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet	Feet.
1871...	3.85	4.45	4.03	3.53	3.00	2.88	2.81	2.75	2.65	2.72	2.79	3.53	3.25
1872...	3.85	3.96	4.08	4.18	3.53	3.15	2.88	2.71	2.55	2.65	2.80	3.10	3.29
1873...	3.20	3.65	3.62	3.61	3.01	2.71	2.42	2.24	2.18	2.28	2.42	2.72	2.84
1874...	3.17	3.19	3.23	3.13	3.06	2.86	2.48	2.39	2.29	2.23	2.41	2.72	2.76
1875...	3.04	3.08	3.04	3.04	2.82	2.46	2.47	2.38	2.15	2.30	2.44	2.64	2.65
1876...	2.84	3.05	3.14	3.11	2.57	1.89	1.50	1.39	1.50	1.83	1.99	2.27	2.26
1877...	2.63	2.87	3.13	3.21	3.22	3.00	2.62	2.56	2.72	2.72	2.93	3.00	2.88
1878...	3.12	3.00	3.77	3.80	3.53	3.25	3.18	3.30	3.47	3.40	3.60	3.92	3.45
1879...	3.82	3.86	3.56	3.95	4.31	4.08	3.84	3.72	3.83	3.74	3.82	4.12	3.89
1880...	4.44	4.48	4.57	4.54	3.85	3.02	2.87	2.88	2.88	2.93	2.99	3.25	3.56
1881...	3.51	3.61	3.70	3.79	3.49	3.05	2.99	2.94	2.71	2.37	2.44	2.72	3.11
1882...	3.07	3.32	3.43	3.51	3.35	3.33	2.88	2.76	2.72	2.89	2.91	3.10	3.11
1883...	3.33	3.62	3.62	3.37	3.36	3.26	3.01	2.99	3.03	3.23	3.38	3.49	3.31
1884...	3.56	3.59	3.71	4.00	3.78	3.58	3.44	3.43	3.16	2.80	2.90	3.11	3.42
1885...	3.34	3.52	3.60	3.65	3.32	3.04	2.80	2.68	2.75	2.92	3.07	3.40	3.17
1886...	3.60	3.73	3.79	3.70	3.45	3.31	3.24	3.33	3.35	3.25	3.40	3.54	3.47
1887...	3.85	3.83	3.52	3.35	3.56	3.40	3.12	3.04	3.18	3.25	3.49	3.71	3.44
1888...	3.82	3.81	3.88	3.88	3.41	2.63	2.44	2.30	2.35	2.44	2.58	2.93	3.04
1889...	3.25	3.47	3.64	3.63	3.28	3.16	2.97	2.78	2.65	2.81	3.12	3.42	3.18
1890...	3.56	3.69	3.93	3.96	3.75	3.30	3.00	2.85	2.72	2.75	2.96	3.32	3.32
1891...	3.68	3.81	3.85	3.89	3.69	3.64	3.44	3.37	3.43	3.38	3.49	3.90	3.63
1892...	3.90	4.18	4.31	4.30	3.97	3.59	3.56	3.44	3.39	3.49	3.66	3.94	3.81
1893...	4.22	4.31	4.26	4.16	3.66	3.14	2.84	2.78	2.87	2.90	3.06	3.29	3.46
1894...	3.47	3.65	3.56	3.41	2.63	2.41	2.35	2.22	2.30	2.28	2.33	2.52	2.76
1895...	2.82	3.04	3.21	3.31	2.94	2.62	2.42	2.37	2.23	2.18	2.47	2.80	2.70
Mean 25 yrs. 1871-95	3.48	3.63	3.69	3.68	3.38	3.07	2.86	2.78	2.76	2.79	2.94	3.22	3.19

TABLE NO. 1—(*Continued*).

Lake Michigan, at Milwaukee, Wis.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 584.34 ft.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	3.12	3.20	2.60	2.40	2.05	2.01	1.98	2.21	2.88	3.57	3.62	4.21	2.82
1872...	4.34	4.34	4.56	4.31	4.06	3.69	3.66	3.68	3.75	3.87	4.16	4.82	4.10
1873...	4.82	4.78	4.47	3.90	3.34	2.71	2.75	2.65	2.84	2.90	3.13	3.17	3.45
1874...	3.21	2.92	2.77	2.87	2.89	2.52	2.59	2.58	2.83	3.18	3.38	3.72	2.96
1875...	3.92	3.99	3.93	3.57	3.01	2.77	2.80	2.63	2.70	2.85	3.06	3.25	3.21
1876...	3.30	3.10	2.77	2.57	1.95	1.54	1.20	1.27	1.32	1.90	1.80	2.27	2.08
1877...	2.41	2.40	2.40	2.02	2.13	2.06	2.09	2.21	2.42	2.41	2.53	2.59	2.31
1878...	2.71	2.78	2.62	2.60	2.30	2.16	2.15	2.47	2.67	2.78	2.91	3.23	2.62
1879...	3.54	3.53	3.49	3.50	3.37	3.30	3.21	3.40	3.52	3.74	3.96	3.93	3.54
1880...	3.89	3.98	3.94	3.77	3.43	2.89	2.67	2.64	2.94	3.28	3.60	3.77	3.40
1881...	3.76	3.55	3.26	2.35	2.84	2.61	2.64	2.64	2.87	2.54	2.71	2.81	2.88
1882...	3.03	3.04	2.67	2.54	2.44	2.17	2.11	1.92	2.04	2.45	2.66	2.99	2.51
1883...	3.25	3.21	3.12	2.91	2.43	2.07	1.47	1.50	1.69	1.91	2.36	2.44	2.36
1884...	2.66	2.54	2.29	2.11	1.90	1.74	1.90	2.04	2.29	2.29	2.65	2.68	2.26
1885...	2.67	2.44	2.48	2.29	1.93	1.72	1.63	1.42	1.56	1.70	2.00	2.29	2.01
1886...	2.06	2.04	1.76	1.49	1.23	1.16	1.35	1.58	1.82	1.92	2.26	2.59	1.77
1887...	2.67	2.30	2.14	2.19	1.99	1.86	1.92	2.06	2.40	2.85	3.18	3.30	2.41
1888...	3.48	3.53	3.35	3.14	2.76	2.49	2.48	2.60	2.75	3.00	3.05	3.63	3.02
1889...	3.65	3.68	3.70	3.69	3.61	3.15	2.97	3.21	3.38	3.63	3.98	4.16	3.57
1890...	4.08	4.12	4.14	3.82	3.59	3.18	3.11	3.19	3.39	3.50	3.84	4.19	3.68
1891...	4.21	4.45	4.26	3.95	3.85	3.70	3.87	3.94	4.17	4.53	4.93	4.99	4.24
1892...	4.87	4.68	4.78	4.72	4.30	3.85	3.84	3.76	3.96	4.20	4.47	4.74	4.35
1893...	4.75	4.61	4.50	4.04	3.74	3.41	3.39	3.56	3.88	4.02	4.41	4.48	4.07
1894...	4.47	4.44	4.18	4.03	3.49	3.33	3.30	3.38	3.81	4.02	4.29	4.64	3.95
1895...	4.82	4.93	4.96	4.76	4.60	4.55	4.66	4.78	5.05	5.42	5.64	5.75	4.99
Mean 25 yrs. 1871-95	3.59	3.54	3.41	3.18	2.93	2.67	2.63	2.68	2.92	3.14	3.39	3.63	3.14

TABLE No. 1—(*Continued*).

Lake Erie, at Cleveland, Ohio.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 575.20 ft.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	2.66	2.99	2.54	2.06	1.79	1.76	1.78	1.99	2.16	2.83	3.01	3.45	2.42
1872...	3.53	3.77	3.86	3.66	3.22	2.85	2.86	2.89	3.12	3.29	3.62	3.85	3.38
1873...	3.95	3.94	3.87	2.59	1.92	1.84	1.86	1.92	2.32	2.62	2.82	2.45	2.67
1874...	2.06	2.01	1.98	1.81	1.72	1.65	1.62	1.78	2.24	2.68	3.10	3.31	2.16
1875...	3.54	3.71	3.57	3.17	2.70	2.27	2.14	2.15	2.29	2.78	2.93	2.71	2.83
1876...	2.75	2.19	1.54	1.02	0.70	0.59	0.70	1.00	1.17	1.70	1.62	1.96	1.41
1877...	2.36	2.52	2.75	2.32	2.07	1.99	1.75	1.89	1.97	2.37	2.45	2.37	2.23
1878...	2.29	2.15	2.02	1.60	1.34	1.34	1.34	1.58	1.71	2.06	2.26	2.18	1.82
1879...	2.60	2.74	2.71	2.35	2.20	2.11	2.08	2.30	2.63	2.86	3.33	3.07	2.58
1880...	2.57	2.53	2.39	2.23	1.96	1.85	1.76	2.00	2.23	2.67	2.75	3.09	2.34
1881...	3.50	3.39	3.07	2.37	1.97	1.73	1.78	2.10	2.45	2.50	2.68	2.47	2.50
1882...	2.00	2.00	1.55	1.33	1.13	0.98	1.05	1.19	1.46	1.91	2.23	2.74	1.63
1883...	2.83	2.62	2.43	2.31	1.85	1.15	0.95	1.01	1.32	1.64	2.02	1.99	1.84
1884...	2.32	2.06	1.87	1.32	1.05	0.97	1.19	1.35	1.78	2.11	2.59	2.66	1.77
1885...	2.84	3.05	3.19	2.37	1.64	1.13	1.17	1.16	1.31	1.41	1.53	1.58	1.87
1886...	1.56	2.29	2.48	1.60	1.30	1.20	1.22	1.43	1.67	1.90	2.31	2.26	1.77
1887...	2.49	2.07	1.26	1.24	1.06	1.03	1.27	1.59	1.82	2.41	2.68	2.66	1.80
1888...	2.84	3.11	3.01	2.38	2.13	2.00	1.85	1.95	2.39	2.76	2.70	2.82	2.50
1889...	2.80	2.96	3.12	2.77	2.59	2.16	1.96	2.27	2.66	3.08	3.35	3.09	2.73
1890...	2.73	2.44	2.32	1.83	1.49	1.12	1.50	1.94	2.13	2.32	2.35	2.58	2.06
1891...	2.80	2.82	2.36	2.49	2.67	2.53	2.63	2.90	3.08	3.46	3.90	3.83	2.96
1892...	3.80	4.01	3.97	3.41	2.61	1.85	1.73	2.08	2.40	2.96	3.29	3.56	2.97
1893...	3.94	3.86	3.64	2.91	2.07	1.88	2.16	2.50	2.88	3.23	3.63	3.55	3.02
1894...	3.27	3.39	3.36	2.96	2.57	2.26	2.38	2.75	2.92	3.24	3.48	3.55	3.01
1895...	3.88	4.11	4.10	3.85	3.63	3.54	3.65	3.73	3.83	4.31	4.41	4.25	3.94
Mean 25 yrs. 1871–95	2.88	2.91	2.76	2.32	1.96	1.75	1.78	1.98	2.24	2.60	2.84	2.88	2.41

TABLE NO. 1—(*Continued*).

Lake Ontario, at Oswego, New York.

Monthly mean of water levels above the zero of the United States Engineer gauge, the elevation of which zero is 244.21 ft. above mean tide at New York City.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	1.94	1.77	1.98	2.58	3.00	2.94	2.78	2.34	2.00	1.50	1.09	0.78	2.06
1872...	0.61	0.39	0.23	0.72	0.84	1.17	1.23	1.07	0.78	0.62	0.57	0.23	0.71
1873...	0.19	0.26	0.38	2.34	2.88	2.80	2.76	2.48	2.06	1.61	1.48	1.67	1.74
1874...	2.23	2.63	3.18	3.07	3.05	3.14	3.09	2.86	2.22	1.82	1.25	0.91	2.45
1875...	0.61	0.26	0.52	1.32	1.59	1.75	1.78	1.64	1.43	1.15	0.96	0.78	1.15
1876...	1.19	1.85	2.40	3.38	3.96	4.18	4.25	3.79	3.18	2.84	2.48	2.30	2.98
1877...	1.77	1.50	1.65	2.34	2.41	2.31	2.35	2.08	1.65	1.22	1.13	1.26	1.81
1878...	1.36	1.57	2.27	2.52	2.86	2.85	2.81	2.73	2.47	2.21	2.09	2.90	2.39
1879...	2.69	2.33	2.18	2.59	2.68	2.71	2.55	2.20	1.78	1.34	0.95	0.98	2.08
1880...	1.20	1.48	1.82	2.00	2.15	2.39	2.40	1.97	1.60	1.19	1.15	0.98	1.69
1881...	0.62	0.61	1.27	1.69	1.87	2.09	2.16	1.84	1.28	1.06	1.06	1.06	1.38
1882...	1.61	1.78	2.38	2.71	2.90	3.41	3.40	3.07	2.69	2.18	1.76	1.47	2.45
1883...	1.20	1.26	1.50	2.02	2.67	3.37	3.90	3.72	3.24	2.80	2.57	2.43	2.56
1884...	2.39	2.76	3.44	4.05	4.07	3.97	3.76	3.53	3.10	2.68	2.18	2.03	3.16
1885...	2.02	1.75	1.47	2.15	2.95	3.32	3.46	3.31	3.09	2.90	2.95	3.12	2.71
1886...	3.48	3.55	3.69	4.31	4.52	4.32	3.92	3.48	3.12	2.83	2.39	2.30	3.49
1887...	2.05	2.80	3.31	3.52	4.08	4.04	3.76	3.25	2.64	2.25	1.90	1.63	2.94
1888...	1.32	1.18	1.42	2.05	2.12	2.16	2.22	2.12	1.73	1.37	1.30	1.29	1.69
1889...	1.50	1.64	1.81	2.05	2.20	2.51	2.70	2.45	1.89	1.45	1.05	1.62	1.91
1890...	2.13	2.48	2.81	3.05	3.41	4.04	3.87	3.20	2.85	2.52	2.60	2.39	2.95
1891...	2.07	2.33	2.87	3.35	3.13	2.71	2.43	1.99	1.56	0.92	0.32	0.29	2.00
1892...	0.39	0.36	0.49	1.07	1.13	1.69	2.20	2.12	1.92	1.48	1.21	1.08	1.26
1893...	0.75	0.64	1.12	1.87	3.03	3.25	2.99	2.45	2.18	1.66	1.25	1.10	1.86
1894...	1.44	1.62	1.92	1.97	2.15	2.68	2.48	1.90	1.39	1.14	0.81	0.46	1.66
1895...	0.37	0.31	0.21	0.76	0.88	0.76	0.47	+0.22	—0.12	—0.46	—0.71	—0.68	0.17
Mean 25 yrs. 1871–95	1.49	1.56	1.85	2.38	2.66	2.82	2.79	2.47	2.07	1.69	1.43	1.38	2.05

TABLE No. 2.—Monthly Changes in Water Level of the Great
Cubic Feet per Second per Month, and the Cum-

By cumulative effect is here meant the increase or decrease in the
level in the

	Superior.		Michigan Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1871.											
January	−0.09	− 30	+0.15	+ 71	+ 41	−0.21	− 22	+ 19	−0.07	− 5	+ 14
February	−0.60	−209	−0.08	− 40	−249	−0.33	− 36	−285	−0.17	− 14	−299
March	+0.42	+146	+0.60	+299	+445	+0.45	+ 49	+494	+0.21	− 17	+511
April	+0.50	+168	+0.20	+ 96	+264	+0.48	+ 51	+315	+0.60	+ 47	+302
May	+0.53	+178	+0.35	+169	+347	+0.27	+ 29	+376	+0.42	+ 33	+409
June	+0.12	+ 40	+0.04	+ 19	+ 59	+0.03	+ 3	+ 62	−0.06	− 5	+ 57
July	+0.07	+ 24	+0.03	+ 14	+ 38	−0.02	− 2	+ 36	−0.16	− 13	+ 23
August	+0.06	+ 20	−0.23	−109	− 89	−0.21	− 22	−111	−0.44	− 34	−145
September	+0.10	+ 34	−0.67	−323	−289	−0.17	− 18	−307	−0.34	− 27	−334
October	−0.07	− 24	−0.69	−333	−357	−0.67	− 71	−428	0.50	− 39	−467
November	−0.07	− 24	−0.05	− 24	− 48	−0.18	− 19	− 67	−0.41	− 32	− 99
December	−0.74	−249	−0.59	−285	−534	−0.44	− 47	−581	−0.31	− 24	−605
1872.											
January	−0.32	−106	−0.13	− 62	−168	−0.08	− 8	−176	−0.17	− 13	−189
February	−0.11	− 38	0.00	00	− 38	−0.24	− 26	− 64	−0.22	− 17	− 81
March	−0.12	− 41	−0.22	−108	−149	−0.09	− 10	−159	−0.16	− 13	−172
April	−0.10	− 34	+0.25	+121	+ 87	+0.20	+ 21	+108	+0.49	+ 39	+147
May	+0.65	+219	+0.25	+121	+340	+0.44	+ 47	+387	+0.12	+ 9	+396
June	+0.38	+128	+0.37	+178	+306	+0.37	+ 39	+345	+0.33	+ 26	+371
July	+0.27	+ 91	+0.03	+ 14	+105	−0.01	− 1	+104	+0.06	+ 5	+109
August	+0.17	+ 56	−0.02	− 9	+ 47	−0.03	− 3	+ 44	−0.16	− 12	+ 32
September	+0.16	+ 54	−0.07	− 34	+ 20	−0.23	− 24	− 4	−0.29	− 23	− 27
October	−0.10	− 34	−0.12	− 58	− 92	−0.17	− 18	−110	−0.16	− 13	−123
November	−0.15	− 50	−0.29	−140	−190	−0.33	− 35	−225	−0.05	− 4	−229
December	−0.30	−101	−0.66	−318	−419	−0.23	− 24	−443	−0.34	− 27	−470
1873.											
January	−0.10	− 33	0.00	000	− 33	−0.10	− 10	− 43	−0.04	− 3	− 46
February	−0.45	−156	+0.04	+ 20	−136	+0.01	+ 1	−135	+0.07	+ 6	−129
March	+0.03	+ 10	+0.31	+155	+165	+0.07	+ 8	+173	+0.12	+ 10	+183
April	+0.01	+ 3	+0.57	+275	+278	+1.28	+135	+413	+1.96	+154	+567
May	+0.60	+202	+0.56	+270	+472	+0.67	+ 71	+543	+0.54	+ 43	+586
June	+0.30	+101	+0.63	+304	+405	+0.08	+ 8	+413	−0.08	− 6	+407
July	+0.29	+ 98	−0.04	− 19	+ 79	−0.02	− 2	+ 77	−0.04	− 3	+ 74
August	+0.18	+ 60	+0.10	+ 47	+107	−0.06	− 6	+101	−0.28	− 22	+ 79
September	+0.06	+ 20	−0.19	− 92	− 72	−0.40	− 42	−114	−0.42	− 33	−147
October	−0.10	− 34	−0.06	− 29	− 63	−0.30	− 32	− 95	−0.45	− 35	−130
November	−0.14	− 47	−0.23	−111	−158	−0.20	− 21	−179	−0.13	− 10	−189
December	−0.30	−101	−0.04	− 19	−120	+0.37	+ 39	− 81	+0.19	+ 15	− 66
1874.											
January	−0.45	−149	−0.04	− 19	−168	+0.39	+ 41	−127	+0.56	+ 43	− 84
February	−0.02	− 7	+0.29	+145	+138	+0.05	+ 5	+143	+0.40	+ 33	+176
March	−0.04	− 14	+0.15	+ 75	+ 61	+0.03	+ 3	+ 64	+0.55	+ 45	+109
April	+0.10	+ 34	−0.10	− 48	− 14	+0.17	+ 18	+ 4	−0.11	− 9	− 5
May	+0.07	+ 24	−0.02	− 10	+ 14	+0.09	+ 10	+ 24	−0.02	− 2	+ 22
June	+0.20	+ 67	+0.37	+178	+245	+0.07	+ 7	+252	+0.09	+ 7	+259
July	+0.38	+128	−0.07	− 34	+ 94	+0.03	+ 3	+ 97	−0.05	− 4	+ 93
August	+0.09	+ 30	+0.01	+ 5	+ 35	−0.16	− 17	+ 18	−0.23	− 18	000
September	+0.10	+ 34	−0.25	−121	− 87	−0.46	− 49	−136	−0.64	− 50	−186
October	+0.06	+ 20	−0.35	−169	−149	−0.44	− 47	−196	−0.40	− 32	−228
November	−0.18	− 61	−0.20	− 96	−157	−0.42	− 44	−201	−0.57	− 45	−246
December	−0.31	−104	−0.34	−164	−268	−0.21	− 22	−290	−0.34	− 27	−317

LAKES, WITH CORRESPONDING RATES OF STORAGE IN THOUSANDS OF
ULATIVE EFFECTS IN THE LAKES BELOW SUPERIOR.

normal discharge of any outlet by the elimination of oscillations of lakes above.

	SUPERIOR.		MICHIGAN-HURON.			ERIE.			ONTARIO.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan—Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan—Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan—Huron—Erie—Ontario.
1875.											
January	−0.32	−106	−0.20	− 95	−201	−0.23	− 24	−225	−0.30	− 23	−248
February	−0.04	− 14	−0.07	− 35	− 49	−0.17	− 19	− 68	−0.35	− 29	− 97
March	+0.04	+ 14	+0.06	+ 30	+ 44	+0.14	+ 15	+ 59	+0.26	+ 21	+ 80
April	0.00	.....	+0.36	+174	+174	+0.40	+ 42	+216	+0.80	+ 63	+279
May	+0.22	+ 74	+0.56	+270	+344	+0.47	+ 50	+394	+0.27	+ 21	+415
June	+0.36	+121	+0.24	+116	+237	+0.43	+ 45	+282	+0.16	+ 13	+295
July	−0.01	− 3	−0.03	− 14	− 17	+0.13	+ 14	− 3	+0.03	+ 2	− 1
August	+0.09	+ 30	+0.17	+ 81	+111	−0.01	− 1	+110	−0.14	− 11	+ 99
September	+0.23	+ 77	−0.07	− 34	+ 43	−0.14	− 15	+ 28	−0.21	− 17	+ 11
October	−0.15	− 50	−0.15	− 72	−122	−0.49	− 52	−174	−0.28	− 22	−196
November	−0.14	− 47	−0.21	−101	−148	−0.15	− 16	−164	−0.19	− 15	−179
December	−0.20	− 67	−0.19	− 92	−159	+0.22	+ 23	−136	−0.18	− 14	−150
1876.											
January	−0.20	− 66	−0.05	− 24	− 90	−0.04	− 4	− 94	+0.41	+ 32	− 62
February	−0.21	− 72	+0.20	+ 98	+ 26	+0.56	+ 60	+ 86	+0.66	+ 53	+139
March	−0.09	− 30	+0.33	+162	+132	+0.65	+ 70	+202	+0.55	+ 44	+246
April	+0.03	+ 10	+0.20	+ 96	+106	+0.52	+ 55	+161	+0.98	+ 77	+238
May	+0.54	+182	+0.62	+299	+481	+0.32	+ 34	+515	+0.58	+ 46	+561
June	+0.68	+229	+0.41	+198	+427	+0.11	+ 12	+439	+0.22	+ 17	+456
July	+0.39	+131	+0.34	+164	+295	−0.11	− 12	+283	+0.07	+ 6	+289
August	+0.11	+ 36	−0.07	− 33	+ 3	−0.30	− 31	− 28	−0.46	− 36	− 64
September	−0.11	− 37	−0.05	− 24	− 61	−0.17	− 18	− 79	−0.61	− 48	−127
October	−0.33	−111	−0.58	−280	−391	−0.53	− 56	−447	−0.34	− 27	−474
November	−0.16	− 54	+0.10	+ 48	− 6	+0.08	+ 8	+ 2	−0.36	− 28	− 26
December	−0.28	− 94	−0.47	−227	−321	−0.34	− 36	−357	−0.18	− 14	−371
1877.											
January	−0.36	−119	−0.14	− 66	−185	−0.40	− 42	−227	−0.53	− 40	−267
February	−0.24	− 83	+0.01	+ 5	− 78	−0.16	− 17	− 95	−0.27	− 22	−117
March	−0.26	− 90	0.00	.....	− 90	−0.23	− 25	−115	+0.15	+ 12	−103
April	−0.08	− 27	+0.38	+183	+156	+0.43	+ 45	+201	+0.69	+ 54	+255
May	−0.01	− 3	−0.11	− 53	− 56	+0.25	+ 26	− 30	+0.07	+ 6	− 24
June	+0.22	+ 74	+0.07	+ 34	+108	+0.08	+ 8	+116	−0.10	− 8	+108
July	+0.38	+128	−0.03	− 14	+114	+0.24	+ 25	+139	+0.04	+ 3	+142
August	+0.06	+ 20	−0.12	− 57	− 37	−0.14	− 15	− 52	−0.27	− 21	− 73
September	−0.16	− 54	−0.21	−101	−155	−0.08	− 8	−163	−0.43	− 34	−197
October	0.00	.....	+0.01	+ 5	+ 5	−0.40	− 42	− 37	−0.43	− 34	− 71
November	−0.21	− 71	−0.12	− 58	−129	−0.08	− 8	−137	−0.09	− 7	−144
December	−0.07	− 24	−0.06	− 29	− 53	+0.08	+ 8	− 45	+0.13	+ 10	− 35
1878.											
January	−0.12	− 40	−0.12	− 57	− 97	+0.08	+ 8	− 89	+0.10	+ 8	− 81
February	+0.12	+ 42	−0.07	− 35	+ 7	+0.14	+ 15	+ 22	+0.21	+ 17	+ 39
March	−0.77	−268	+0.16	+ 80	−188	+0.13	+ 14	−174	+0.70	+ 57	−117
April	−0.03	− 10	+0.02	+ 10	00	+0.42	+ 44	+ 44	+0.25	+ 20	+ 64
May	+0.27	+ 91	+0.30	+145	+236	+0.26	+ 27	+263	+0.34	+ 27	+290
June	+0.28	+ 94	+0.14	+ 68	+162	0.00	.....	+162	−0.01	− 1	+161
July	+0.07	+ 24	+0.01	+ 5	+ 29	0.00	.....	+ 29	−0.04	− 3	+ 26
August	−0.12	− 40	−0.32	−152	−192	−0.24	− 25	−217	−0.08	− 6	−223
September	−0.17	− 57	−0.20	− 96	−153	−0.13	− 14	−167	−0.26	− 20	−187
October	+0.07	+ 24	−0.11	− 53	− 29	−0.35	− 37	− 66	−0.26	− 20	− 86
November	−0.20	− 67	−0.13	− 63	−130	−0.20	− 21	−151	−0.12	− 9	−160
December	−0.32	−104	−0.32	−159	−263	+0.08	+ 8	−255	+0.81	+ 64	−191

TABLE No. 2—

	Superior.		Michigan Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1879.											
January	+0.10	+ 33	−0.31	−147	−114	−0.42	− 44	−158	−0.21	− 16	−174
February	−0.04	− 14	+0.01	+ 5	− 9	−0.14	− 15	− 24	−0.36	− 29	− 53
March	+0.30	+104	+0.04	+ 20	+124	+0.03	+ 3	+127	−0.15	− 12	+115
April	−0.39	−131	−0.01	− 5	−136	+0.36	+ 38	− 98	+0.41	+ 32	− 66
May	−0.36	−121	+0.13	+ 63	− 58	+0.15	+ 16	− 42	+0.09	+ 7	− 35
June	+0.23	+ 77	+0.07	+ 34	+111	+0.09	+ 10	+121	+0.03	+ 2	+123
July	+0.24	+ 81	+0.09	+ 43	+124	+0.03	+ 3	+127	−0.16	− 13	+114
August	+0.12	+ 40	−0.19	− 90	− 50	−0.22	− 23	− 73	−0.35	− 27	−100
September	−0.11	− 37	−0.12	− 58	− 95	−0.33	− 35	−130	−0.42	− 33	−163
October	+0.09	+ 30	−0.22	−106	− 76	−0.23	− 24	−100	−0.44	− 35	−135
November	−0.08	− 27	−0.22	−106	−133	−0.47	− 50	−183	−0.39	− 31	−214
December	−0.30	−101	+0.03	+ 14	− 87	+0.26	+ 27	− 60	+0.03	+ 2	− 58
1880.											
January	−0.32	−106	+0.04	+ 19	− 87	+0.50	+ 52	− 35	+0.22	+ 17	− 18
February	−0.04	− 14	−0.09	− 44	− 58	+0.04	+ 4	− 54	+0.28	+ 22	− 32
March	−0.09	− 31	+0.04	+ 20	− 11	+0.14	+ 15	+ 4	+0.34	+ 27	+ 31
April	+0.03	+ 10	+0.17	+ 82	+ 92	+0.16	+ 17	+109	+0.18	+ 14	+123
May	+0.69	+232	+0.34	+164	+396	+0.27	+ 29	+425	+0.15	+ 12	+437
June	+0.83	+279	+0.54	+260	+539	+0.11	+ 12	+551	+0.24	+ 19	+570
July	+0.15	+ 50	+0.22	+106	+156	+0.09	+ 10	+166	+0.01	+ 1	+167
August	−0.01	− 3	+0.03	+ 14	+ 11	−0.24	− 25	− 14	−0.43	− 33	− 47
September	0.00	.....	−0.30	−145	−145	−0.23	− 24	−169	−0.37	− 29	−198
October	−0.05	− 17	−0.34	−164	−181	−0.44	− 47	−228	−0.41	− 32	−260
November	−0.06	− 20	−0.32	−154	−174	−0.08	− 8	−182	−0.14	− 11	−193
December	−0.26	− 87	−0.17	− 82	−169	−0.34	− 36	−205	−0.17	− 13	−218
1881.											
January	−0.26	− 86	+0.01	+ 5	− 81	−0.41	− 43	−124	−0.36	− 28	−152
February	−0.10	− 35	+0.21	+105	+ 70	+0.11	+ 12	+ 82	−0.01	− 1	+ 81
March	−0.09	− 31	+0.29	+145	+114	+0.32	+ 35	+149	+0.66	+ 54	+203
April	−0.09	− 30	−0.09	− 43	− 73	+0.70	+ 74	+ 1	+0.42	+ 33	+ 34
May	+0.30	+101	−0.49	−236	−135	+0.40	+ 42	− 93	+0.18	+ 14	− 79
June	+0.44	+148	+0.23	−111	+259	−0.24	− 25	+284	+0.22	+ 17	+301
July	+0.06	+ 20	−0.03	− 14	+ 6	−0.05	− 5	+ 1	+0.07	+ 6	+ 7
August	+0.05	+ 17	0.00	.....	+ 17	−0.32	− 33	− 16	−0.32	− 25	− 41
September	+0.23	+ 77	−0.23	−111	− 34	−0.35	− 37	− 71	−0.56	− 44	−115
October	+0.34	+114	+0.33	+159	+273	−0.05	− 5	+268	−0.22	− 17	+251
November	−0.07	− 24	−0.17	− 82	−106	−0.18	− 19	−125	0.00	.....	−125
December	−0.28	− 94	−0.10	− 48	−142	+0.21	+ 22	−120	0.00	.....	−120
1882.											
January	−0.35	−116	−0.22	−104	−220	+0.47	+ 49	−171	+0.55	+ 43	−128
February	−0.25	− 87	−0.01	− 5	− 92	0.00	.....	− 92	+0.17	+ 14	− 78
March	−0.11	− 38	+0.37	+184	+146	+0.45	+ 49	+195	+0.60	+ 49	+244
April	−0.08	− 27	+0.13	+ 63	+ 36	+0.22	+ 23	+ 59	+0.33	+ 26	+ 85
May	+0.16	+ 54	+0.10	+ 48	+102	+0.20	+ 21	+123	+0.19	+ 15	+138
June	+0.02	+ 7	+0.27	+130	+137	+0.15	+ 16	+153	+0.51	+ 40	+193
July	+0.45	+151	+0.06	+ 29	+180	−0.07	− 7	+173	−0.01	− 1	+172
August	+0.12	+ 40	+0.19	+ 90	+130	−0.14	− 15	+115	−0.33	− 26	+ 89
September	+0.04	+ 13	−0.12	− 58	− 45	−0.27	− 29	− 74	−0.38	− 30	−104
October	−0.17	− 57	−0.41	−198	−255	−0.45	− 48	−303	−0.51	− 40	−343
November	−0.02	− 7	−0.21	−101	−108	−0.32	− 34	−142	−0.42	− 33	−175
December	−0.19	− 64	−0.33	−159	−223	−0.51	− 54	−277	−0.29	− 23	−300
1883.											
January	−0.23	− 76	−0.26	−123	−199	−0.09	− 9	−208	−0.27	− 21	−229
February	−0.29	−101	+0.04	+ 20	− 81	+0.21	+ 23	− 58	+0.06	+ 5	− 53
March	0.00	.....	+0.09	− 45	+ 45	+0.19	+ 21	+ 66	+0.24	+ 20	+ 86
April	+0.25	+ 84	+0.21	+101	+185	+0.12	+ 13	+198	+0.52	+ 41	+239
May	+0.01	+ 3	+0.48	+231	+234	+0.46	+ 49	+283	+0.65	+ 51	+334
June	+0.10	+ 34	+0.36	+174	+208	+0.70	+ 74	+282	+0.70	+ 55	+337
July	+0.25	+ 84	+0.60	+289	+373	+0.20	+ 21	+394	+0.53	+ 42	+436
August	+0.02	+ 7	−0.03	− 14	− 7	−0.06	− 6	− 13	−0.18	− 14	− 27
September	−0.04	− 13	−0.19	− 92	−105	−0.31	− 33	−138	−0.48	− 38	−176
October	−0.20	− 67	−0.22	−106	−173	−0.32	− 34	−207	−0.44	− 35	−242
November	−0.15	− 50	−0.45	−217	−267	−0.38	− 40	−307	−0.23	− 18	−325
December	−0.11	− 37	−0.08	− 39	− 76	+0.03	+ 3	− 73	−0.14	− 11	− 84

(*Continued*).

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron—Erie—Ontario.
1884.											
January	−0.07	− 23	−0.22	−104	−127	−0.33	− 34	−161	−0.04	− 3	−164
February	−0.03	− 10	+0.12	+ 59	+ 49	+0.26	+ 28	+ 77	+0.37	+ 30	+107
March	−0.12	− 41	+0.25	+123	+ 82	+0.19	+ 20	+102	+0.68	+ 54	+156
April	−0.29	− 98	+0.18	+ 87	− 11	+0.55	+ 58	+ 47	+0.61	+ 48	+ 95
May	+0.22	+ 74	+0.21	+101	+175	+0.27	+ 29	+204	+0.02	+ 2	+206
June	+0.20	+ 67	+0.16	+ 77	+144	+0.08	+ 8	+152	−0.10	− 8	+144
July	+0.14	+ 47	−0.16	− 77	− 30	−0.22	− 23	− 53	−0.21	− 17	− 70
August	+0.01	+ 3	−0.14	− 66	− 63	−0.16	− 17	− 80	−0.23	− 18	− 98
September	+0.27	+ 91	−0.25	−121	− 30	−0.43	− 45	− 75	−0.43	− 34	−109
October	+0.36	+121	0.00	.....	+121	−0.33	− 35	+ 86	−0.42	− 33	+ 53
November	−0.10	− 34	−0.36	−174	−208	−0.48	− 51	−259	−0.50	− 39	−298
December	−0.21	− 71	−0.03	− 14	− 85	−0.07	− 7	− 92	−0.15	− 12	−104
1885.											
January	−0.23	− 76	+0.01	+ 5	− 71	−0.18	− 19	− 90	−0.01	− 1	− 91
February	−0.18	− 63	+0.23	+115	+ 52	−0.21	− 23	+ 29	−0.27	− 22	+ 7
March	−0.08	− 28	−0.04	− 20	− 48	−0.14	− 15	− 63	−0.28	− 23	− 86
April	−0.05	− 17	+0.19	+ 92	+ 75	+0.82	+ 87	+162	+0.68	+ 54	+216
May	+0.33	+111	+0.36	+174	+285	+0.73	+ 77	+362	+0.80	+ 63	+425
June	+0.28	+ 94	+0.21	+101	+195	+0.51	+ 54	+249	+0.37	+ 29	+278
July	+0.24	+ 81	+0.09	+ 43	+124	−0.04	− 4	+120	+0.14	+ 11	+131
August	+0.12	+ 40	+0.21	+100	+140	+0.01	+ 1	+141	−0.15	− 12	+129
September	−0.07	− 24	−0.14	− 68	− 92	−0.15	− 16	−108	−0.22	− 17	−125
October	−0.17	− 57	−0.14	− 68	−125	−0.10	− 11	−136	−0.19	− 15	−151
November	−0.15	− 50	−0.30	−145	−195	−0.12	− 13	−208	+0.05	+ 4	−204
December	−0.33	−111	−0.29	−140	−251	−0.05	− 5	−256	+0.17	+ 13	−243
1886.											
January	−0.20	− 66	+0.23	+109	− 43	+0.02	− 2	+ 45	+0.36	+ 28	+ 73
February	−0.13	− 45	+0.02	+ 10	− 35	−0.73	− 80	−115	+0.07	+ 6	−109
March	−0.06	− 21	+0.28	+140	+119	−0.19	− 21	+ 98	+0.14	+ 11	+109
April	+0.09	+ 30	+0.27	+130	+160	+0.88	+ 93	+253	+0.62	+ 49	+302
May	+0.25	+ 84	+0.26	+125	+209	+0.30	+ 32	+241	+0.21	+ 17	+258
June	+0.14	+ 47	+0.07	+ 34	+ 81	−0.10	+ 11	+ 92	−0.20	− 16	+ 76
July	+0.07	+ 24	−0.19	− 92	− 68	−0.02	− 2	− 70	−0.40	− 32	−102
August	−0.09	− 30	−0.23	−109	−139	−0.21	− 22	−161	−0.44	− 34	−195
September	−0.02	− 7	−0.24	−116	−123	−0.24	− 25	−148	−0.36	− 28	−176
October	+0.10	+ 34	−0.10	− 48	− 14	−0.23	− 24	− 38	−0.29	− 23	− 61
November	−0.15	− 50	−0.34	−164	−214	−0.41	− 43	−257	−0.44	− 35	−292
December	−0.14	− 47	−0.33	−159	−206	+0.05	− 5	−201	−0.09	− 7	−208
1887.											
January	−0.31	−103	−0.08	− 38	−141	−0.23	− 24	−165	−0.25	− 19	−184
February	+0.02	+ 7	+0.37	+184	+191	+0.42	+ 46	+237	+0.75	+ 61	+298
March	+0.31	+108	+0.16	+ 80	+188	+0.81	+ 89	+277	+0.51	+ 42	+319
April	+0.17	+ 57	−0.05	− 24	+ 33	+0.02	+ 2	+ 35	+0.21	+ 17	+ 52
May	−0.21	− 71	+0.20	+ 96	+ 25	+0.18	+ 19	+ 44	+0.56	+ 44	+ 88
June	+0.16	+ 54	+0.13	+ 63	+117	+0.03	+ 3	+120	−0.04	− 3	+117
July	+0.28	+ 94	−0.06	− 29	+ 65	−0.24	− 25	+ 40	−0.28	− 22	+ 18
August	+0.08	+ 26	−0.14	− 66	− 40	−0.32	− 33	− 73	−0.51	− 40	−113
September	−0.14	− 47	−0.34	−164	−211	−0.23	− 24	−235	−0.61	− 48	−283
October	−0.07	− 24	−0.45	−217	−241	−0.59	− 62	−303	−0.39	− 31	−334
November	−0.24	− 81	−0.33	−159	−240	−0.27	− 29	−269	−0.35	− 27	−296
December	−0.22	− 74	−0.12	− 58	−132	+0.02	+ 2	−130	−0.27	− 21	−151
1888.											
January	−0.11	− 36	−0.18	− 85	−121	−0.18	− 19	−140	−0.31	− 24	−164
February	+0.01	+ 3	−0.05	− 25	− 22	−0.27	− 29	− 51	−0.14	− 11	− 62
March	−0.07	− 24	+0.18	+ 88	+ 64	+0.10	+ 11	+ 75	+0.24	+ 19	+ 94
April	0.00	.....	+0.21	+101	+101	+0.63	+ 67	+168	+0.63	+ 50	+218
May	+0.47	+158	+0.38	+183	+341	+0.25	+ 26	+367	+0.07	+ 6	+373
June	+0.78	+262	+0.27	+130	+392	+0.13	+ 14	+406	+0.04	+ 3	+409
July	+0.19	+ 64	+0.01	+ 5	+ 69	+0.15	+ 16	+ 85	+0.06	+ 5	+ 90
August	+0.14	+ 46	−0.12	− 57	− 11	−0.10	− 10	− 21	−0.10	− 8	− 29
September	−0.05	− 17	−0.15	− 72	− 89	−0.44	− 47	−136	−0.39	− 31	−167
October	−0.09	− 30	−0.25	−121	−151	−0.37	− 39	−190	−0.36	− 28	−218
November	−0.14	− 47	−0.05	− 24	− 71	+0.06	+ 6	− 65	−0.07	− 6	− 71
December	−0.35	−118	−0.58	−280	−398	−0.12	− 13	−412	−0.01	− 1	−413

TABLE No. 2—

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1889.											
January	−0.32	−106	−0.02	− 9	−115	+0.02	+ 2	−113	+0.21	+ 16	− 97
February	−0.22	− 77	−0.03	− 15	− 92	−0.16	− 17	−109	+0.14	+ 11	− 98
March	−0.17	− 59	−0.02	− 10	− 69	−0.16	− 17	− 86	+0.17	+ 14	− 72
April	+0.01	+ 3	+0.01	+ 5	+ 8	+0.35	+ 37	+ 45	+0.24	+ 19	+ 64
May	+0.35	+118	+0.08	+ 39	+157	+0.18	+ 19	+176	+0.15	+ 12	+188
June	+0.12	+ 40	+0.46	+222	+262	+0.43	+ 45	+307	+0.31	+ 24	+331
July	+0.19	+ 64	+0.18	+ 87	+151	+0.20	+ 21	+172	+0.19	+ 15	+187
August	+0.19	+ 63	−0.24	−114	− 51	−0.31	− 32	− 83	−0.25	− 19	−102
September	+0.13	+ 44	−0.17	− 82	− 38	−0.39	− 41	− 79	−0.56	− 44	−123
October	−0.16	− 54	−0.25	−121	−175	−0.42	− 44	−219	−0.44	− 35	−254
November	−0.31	−104	−0.35	−169	−273	−0.27	− 29	−302	−0.40	− 32	−334
December	−0.30	−101	−0.18	− 87	−188	+0.26	+ 27	−161	+0.57	+ 45	−116
1890.											
January	−0.14	− 46	+0.08	+ 38	− 8	+0.36	+ 37	+ 29	+0.51	+ 40	+ 69
February	−0.13	− 45	−0.04	− 20	− 65	+0.29	+ 32	− 33	+0.35	+ 29	− 4
March	−0.24	− 83	−0.02	− 10	− 93	+0.12	+ 13	− 80	+0.33	+ 27	− 53
April	−0.03	− 10	+0.32	+154	+144	+0.49	+ 52	+196	+0.24	+ 19	+215
May	+0.21	+ 71	+0.23	+111	+182	+0.34	+ 36	+218	+0.36	+ 28	+246
June	+0.45	+151	+0.41	+198	+349	+0.37	+ 39	+388	+0.63	+ 50	+438
July	+0.30	+101	+0.07	+ 34	+135	−0.38	− 40	+ 95	−0.17	− 13	+ 82
August	+0.15	+ 50	−0.08	− 38	+ 12	−0.44	− 46	− 34	−0.67	− 52	− 86
September	+0.13	+ 44	−0.20	− 96	− 52	−0.19	− 20	− 72	−0.35	− 28	−100
October	−0.03	− 10	−0.11	− 53	− 63	−0.19	− 20	− 83	−0.33	− 26	−109
November	−0.21	− 71	−0.34	−164	−235	−0.03	− 3	−238	+0.08	+ 6	−232
December	−0.36	−121	−0.35	−169	−290	−0.23	− 24	−314	−0.21	− 17	−331
1891.											
January	−0.36	−119	−0.02	− 9	−128	−0.22	− 23	−151	−0.32	− 25	−176
February	−0.13	− 45	−0.24	−120	−165	−0.02	− 2	−167	+0.26	+ 21	−146
March	−0.04	− 14	+0.19	+ 95	+ 81	+0.46	+ 50	+131	+0.54	+ 44	+175
April	−0.04	− 13	+0.31	+150	+137	−0.13	− 14	+123	+0.48	+ 38	+161
May	+0.20	+ 67	+0.10	+ 48	+115	−0.18	− 19	+ 96	−0.22	− 17	+ 79
June	+0.05	+ 17	+0.15	+ 72	+ 89	+0.14	+ 15	+104	−0.42	− 33	+ 71
July	+0.20	+ 67	−0.17	− 82	− 15	−0.10	− 11	− 26	−0.28	− 22	− 48
August	+0.07	+ 23	−0.07	− 33	− 10	−0.27	− 28	− 38	−0.44	− 34	− 72
September	−0.06	− 20	−0.23	−111	−131	−0.18	− 19	−150	−0.43	− 34	−184
October	+0.05	+ 17	−0.36	−174	−157	−0.38	− 40	−197	−0.64	− 50	−247
November	−0.11	− 37	−0.40	−193	−230	−0.44	− 47	−277	−0.60	− 47	−324
December	−0.41	−139	−0.06	− 29	−168	+0.07	+ 7	−161	−0.08	− 2	−163
1892.											
January	0.00	.....	+0.12	+ 57	+ 57	+0.03	+ 3	+ 60	+0.10	+ 8	+ 68
February	−0.28	− 96	+0.19	+ 93	− 3	−0.21	− 23	− 26	−0.03	− 2	− 28
March	−0.13	− 44	−0.10	− 49	− 93	+0.04	+ 4	− 89	+0.13	+ 10	− 79
April	+0.01	+ 3	+0.06	+ 29	− 32	+0.56	+ 58	+ 90	+0.58	+ 46	+136
May	+0.33	+111	+0.42	+203	+314	+0.80	+ 85	+399	+0.06	+ 5	+404
June	+0.38	+128	+0.45	+217	+345	+0.76	+ 80	+425	+0.56	+ 44	+469
July	+0.03	+ 10	+0.01	+ 5	+ 15	+0.12	+ 13	+ 28	+0.51	+ 40	+ 68
August	+0.12	+ 40	+0.08	+ 38	+ 78	−0.35	− 36	+ 42	−0.08	− 6	+ 36
September	+0.05	+ 17	−0.20	− 96	− 79	−0.32	− 34	−113	−0.20	− 16	−129
October	−0.10	− 34	−0.24	−116	−150	−0.56	− 59	−209	−0.44	− 35	−244
November	−0.17	− 57	−0.27	−130	−187	−0.33	− 35	−222	−0.27	− 21	−243
December	−0.28	− 94	−0.27	−130	−224	−0.27	− 29	−253	−0.13	− 10	−263

(*Continued*).

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1893.											
January	−0.28	− 93	−0.01	− 5	− 98	−0.38	− 40	−138	−0.33	− 26	−164
February	−0.09	− 31	+0.14	+ 70	+ 39	+0.08	− 9	− 48	−0.11	− 9	+ 39
March	−0.05	+ 17	+0.11	+ 55	+ 72	+0.22	− 24	+ 96	−0.48	+ 39	−135
April	+0.10	+ 34	+0.46	+222	+256	+0.73	− 77	+333	+0.75	+ 59	+392
May	+0.50	+168	+0.30	−145	+313	+0.84	− 89	+402	+1.16	+ 91	+493
June	+0.52	+175	+0.33	−159	+334	+0.19	+ 20	+354	+0.22	+ 17	+371
July	+0.30	+101	+0.02	+ 10	+111	−0.28	− 30	+ 81	−0.26	− 20	+ 61
August	+0.06	+ 20	−0.17	− 81	− 61	−0.34	− 35	− 96	−0.54	− 42	−138
September	−0.09	− 30	−0.32	−154	−184	−0.38	− 40	−224	−0.27	− 21	−245
October	−0.03	− 10	−0.14	− 68	− 78	−0.35	− 37	−115	−0.52	− 41	−156
November	−0.16	− 54	−0.39	−188	−242	−0.40	− 42	−284	−0.41	− 32	−316
December	−0.23	− 77	−0.07	− 34	−111	+0.08	+ 8	−103	−0.15	− 12	−115
1894.											
January	−0.18	− 60	+0.01	+ 5	− 55	+0.28	+ 29	− 26	−0.34	− 26	0
February	−0.18	− 63	+0.03	+ 15	− 48	−0.12	− 13	− 61	−0.18	− 15	− 46
March	+0.09	+ 31	+0.26	−130	+161	+0.03	+ 3	+164	+0.30	+ 24	+188
April	+0.15	+ 50	+0.15	− 72	+122	+0.40	+ 42	+164	+0.05	+ 4	+168
May	+0.78	+262	+0.54	+260	+522	+0.39	+ 41	+563	+0.18	+ 14	+577
June	+0.22	+ 74	+0.16	− 77	+151	+0.31	+ 33	+184	+0.53	+ 42	+226
July	+0.06	+ 20	+0.03	− 14	+ 34	−0.12	− 13	+ 21	−0.20	− 16	+ 5
August	+0.13	+ 43	−0.08	− 38	− 5	−0.37	− 39	− 34	−0.58	− 45	− 79
September	−0.08	− 27	−0.43	−207	−234	−0.17	− 18	−252	−0.51	− 40	−292
October	+0.02	+ 7	−0.21	−101	− 94	−0.32	− 34	−128	−0.25	− 20	−148
November	−0.05	− 17	−0.27	−130	−147	−0.24	− 25	−172	−0.33	− 26	−198
December	−0.19	− 64	−0.35	−169	−233	−0.07	− 7	−240	−0.35	− 28	−268
1895.											
January	−0.30	− 99	−0.18	− 85	−184	−0.33	− 34	−218	−0.09	− 7	−225
February	−0.22	− 77	−0.11	− 55	−132	−0.23	− 25	−157	−0.06	− 5	−162
March	−0.17	− 59	−0.03	− 15	− 74	+0.01	+ 1	− 73	−0.10	− 8	− 81
April	−0.10	− 34	+0.20	+ 96	+ 62	+0.25	− 26	+ 88	+0.55	+ 43	+131
May	+0.37	+124	+0.16	+ 77	+201	+0.22	+ 23	+224	+0.12	+ 9	+233
June	+0.32	+108	+0.05	+ 24	+132	+0.09	+ 10	+142	−0.12	− 9	+133
July	+0.20	+ 67	−0.11	− 53	+ 14	−0.11	− 12	+ 2	−0.29	− 23	− 21
August	+0.05	+ 17	−0.12	− 57	− 40	−0.08	− 8	− 48	−0.25	− 19	− 67
September	+0.14	+ 47	−0.27	−130	− 83	−0.10	− 11	− 94	−0.34	− 27	−121
October	+0.05	+ 17	−0.37	−178	−161	−0.48	− 51	−212	−0.34	− 27	−239
November	−0.29	− 98	−0.22	−106	−204	−0.10	− 11	−215	−0.25	− 20	−235
December	−0.33	−111	−0.11	− 53	−164	+0.16	+ 17	−147	+0.03	+ 2	−145
Mean for 25 years. 1871–95.											
January	−0.26	− 86	−0.04	− 19	−105	0.00	0	−105	+0.11	− 8	− 97
February	−0.15	− 52	+0.05	+ 25	− 27	−0.03	− 3	− 30	+0.07	+ 6	− 24
March	−0.06	− 21	+0.13	+ 65	+ 44	+0.15	+ 16	+ 60	+0.28	+ 22	+ 82
April	+0.01	+ 3	+0.23	−110	+113	+0.44	+ 46	−159	+0.53	+ 41	+200
May	+0.30	+101	+0.25	+120	+221	+0.36	+ 38	+259	+0.28	+ 21	+280
June	+0.31	+104	+0.26	+125	+229	+0.21	+ 22	+251	+0.16	+ 12	+263
July	+0.21	+ 70	+0.04	+ 19	+ 89	−0.03	− 3	+ 86	−0.03	− 2	− 84
August	+0.08	+ 26	−0.05	− 24	+ 2	−0.20	− 21	− 19	−0.32	− 25	− 44
September	+0.02	+ 7	−0.24	−115	−108	−0.26	− 27	−135	−0.40	− 32	−167
October	−0.03	− 10	−0.22	−106	−116	−0.36	− 38	−154	−0.38	− 30	−184
November	−0.15	− 50	−0.25	−120	−170	−0.24	− 25	−195	−0.26	− 21	−216
December	−0.28	− 94	−0.24	−115	−209	−0.04	− 4	−213	−0.05	− 4	−217

TABLE No. 3.—Oscillations in Mean Annual Levels of the Great
per Second for each Period of One Year. The Table
Effect at each Lake by Adding

Period.	Lake Superior.		Michigan-Huron.		
	Fluctuation.	Storage.	Fluctuation.	Storage for single lake.	Cumulative storage.
1870–71	—0.36	—10.1	—0.07	— 2.8	—12.9
1871–72	—0.04	— 1.1	—1.28	—51.6	—52.7
1872–73	+0.45	+12.6	+0.65	+26.2	+38.8
1873–74	+0.8	+ 2.2	+0.49	+19.8	+22.3
1874–75	+0.11	+ 3.1	—0.25	—10.1	— 7.0
1875–76	+0.39	+11.0	+1.13	+45.5	+56.5
1876–77	—0.62	—17.4	—0.23	— 9.3	—26.7
1877–78	—0.57	—16	—0.31	—12.5	—28.5
1878–79	—0.44	—13.8	—0.92	—37.1	—50.9
1879–80	+0.33	+ 9.3	+0.14	+ 5.6	+14.9
1880–81	+0.45	+12.6	+0.52	+21.0	+33.6
1881–82	0.00	0.0	+0.37	+14.9	+14.9
1882–83	—0.20	— 5.6	+0.15	+ 6.1	+ 0.3
1883–84	—0.11	— 3.1	+0.10	+ 4.0	+ 0.9
1884–85	+0.25	+ 7.0	+0.25	+10.1	+17.1
1885–86	—0.30	— 8.4	+0.24	+ 9.7	+ 1.3
1886–87	+0.03	+ 0.8	—0.64	—25.8	—25.0
1887–88	+0.40	+11.2	—0.61	—24.6	—13.4
1888–89	—0.14	— 3.9	—0.55	—22.2	—26.1
1889–90	—0.14	— 3.9	—0.11	— 4.4	— 8.3
1890–91	—0.31	— 8.7	—0.56	—22.6	—31.3
1891–92	—0.18	— 5.1	—0.11	— 4.4	— 9.5
1892–93	+0.35	+ 9.8	+0.28	+11.3	+21.1
1893–94	+0.70	+19.7	—0.12	— 4.8	+14.9
1894–95	+0.06	+ 1.7	—1.04	—41.9	—40.2

LAKES, AND THE EQUIVALENT STORAGE IN THOUSANDS OF CUBIC FEET GIVES THE VALUES FOR EACH LAKE AND THE CUMULATIVE THE STORAGE IN THOSE ABOVE.

Period.	Erie.			Ontario.		
	Fluctuation.	Storage for single lake.	Cumulative storage.	Fluctuation.	Storage for single lake.	Cumulative storage.
1870-71.....	—0.59	— 5.2	—18.1	—1.41	— 9.3	—27.4
1871-72.....	—0.96	— 8.5	—61.2	—1.35	— 8.9	—70.1
1872-73.....	+0.71	+ 6.3	+45.1	+1.03	+ 6.8	+51.9
1873-74.....	+0.51	+ 4.5	+26.5	+0.71	+ 4.7	+31.2
1874-75.....	—0.67	— 5.9	—12.9	—1.30	— 8.6	—21.5
1875-76.....	+1.42	+12.6	+69.	+1.83	+12.1	+81.2
1976-77.....	—0.82	— 7.2	—33.9	—1.17	— 7.7	—41.6
1877-78.....	+0.41	+ 3.6	—24.9	+0.58	+ 3.8	—21.1
1878-79.....	—0.76	— 6.7	—57.6	—0.31	— 2.0	—59.6
1879-80.....	+0.24	+ 2.1	+17.0	—0.39	— 2.6	—14.4
1880-81.....	—0.16	— 1.4	+32.2	—0.31	— 2.0	+30.2
1881-82.....	+0.87	+ 7.7	+22.6	+1.07	+ 7.0	+33.3
1882-83.....	—0.21	— 1.9	— 1.6	+0.11	+ 0.7	— 0.5
1883-84.....	+0.07	+ 0.6	+ 1.5	+0.60	+ 4.0	+ 5.5
1884-85.....	—0.10	— 0.9	+16.2	—0.45	— 3.0	+13.2
1885-86.....	+0.10	+ 0.9	+ 2.2	+0.78	+ 5.1	+ 7.3
1886-87.....	—0.03	— 0.3	—25.3	—0.55	— 3.6	—28.9
1887-88.....	—0.70	— 6.2	—19.6	—1.25	— 8.2	—27.8
1888-89.....	—0.23	— 2.0	—28.1	+0.22	+ 1.5	—26.6
1889-90.....	+0.67	+ 5.9	— 2.4	+1.04	+ 6.9	+ 4.5
1890-91.....	—0.90	— 8.0	—39.3	—0.95	— 6.3	—45.6
1891-92.....	—0.01	— 0.1	— 9.6	—0.74	— 4.9	—14.5
1892-93.....	—0.05	— 0.4	+20.7	+0.60	+ 4.0	+24.7
1893-94.....	+0.01	+ 0.1	+15.0	—0.20	— 1.3	+13.7
1894-95.....	—0.93	— 8.2	—48.4	—1.49	— 9.8	—58.2

TABLE No. 4.—Storage in Thousands of Cubic Feet per Second Represented by a Uniform Rise or Fall of 1 Ft. and of 6 Ins. for Periods Varying from 1 to 6 Months, in the Great Lakes of the St. Lawrence Basin.

Time.	Lake Superior.		Lake Michigan-Huron.				Lake Erie.				Lake Ontario.			
			1 ft.		6 ins.		1 ft.		6 ins.		1 ft.		6 ins.	
	1 ft.	6 ins.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.
1 month....	336	168	482	818	241	409	106	924	53	462	79	1 003	39	501
2 months...	168	84	241	409	121	204	53	462	26	231	39	501	20	250
3 months...	112	56	161	273	80	136	35	308	18	154	26	334	13	167
4 months...	84	42	121	205	60	102	26	231	13	115	20	251	10	125
5 months...	67	34	96	164	48	82	21	185	11	92	16	201	8	100
6 months...	56	28	80	136	40	68	18	154	9	77	13	167	7	84

A MATHEMATICAL ANALYSIS OF THE INFLUENCE OF RESERVOIRS UPON STREAM-FLOW.

By James A. Seddon,* M. Am. Soc. C. E.

The fluctuation in the levels of a system of lakes only differs in its magnitude from the filling and emptying of a lock or a reservoir under varying conditions of inflow and outflow, though, of course, the true level over such large areas is not so easily observed. The action of the winds will certainly greatly change the levels at different points, the surface will answer to difference in barometric pressure, and small tidal effects may be found, and even after the primary disturbance is removed, the whole body of water may have been set to rocking backward and forward like a pendulum. It is thus only in the mean of a large number of points that the true level may be determined, or, as in actual practice, in the average of a time long enough to eliminate these shorter variations; but, this true level determined, all its variations are results alone of the inflow and the outflow.

The inflow will be called supply, and designated by S, and the outflow, discharge, and designated by Q, each taken in cubic feet per second, and with A representing the area of the lake in square feet, h its level on a scale of feet and T time in seconds. The fundamental equation of its fluctuation is

$$(S - Q)\, d\, T = A\, d\, h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (I)$$

In this equation, S, the supply of the lake, is made up of the run-off from the drainage basin surrounding it, which has its annual rise and fall, like the high and low-water periods of a river in a similar basin. But, unlike the river, the direct rainfall on its large area gives it at times a much more concentrated supply, and undoubtedly now and then its actual levels are subject to finite changes in comparatively insignificant intervals of time; but averaged up, as these are in the mean monthly levels, the effect of this direct supply approaches the annual cycle, and for a number of years must closely correspond to the variation of normal rainfall, and, finally, from these positive elements of supply there must be taken the negative element of evaporation, which has perhaps of all of them the most regular variation with the seasons.

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Thus, while the actual supply in a given year can never be exactly represented by any regular variation, the general type of supply may very fairly be assumed as a recurrent annual oscillation, and taken in its simplest form, as $S_1 + a \sin t$; where S_1 is a mean value of supply and t is a time arc on a circle, whose whole circumference represents one year, while as sin t passes from its extreme values of $+1$ to -1, the extreme oscillation of supply, of course, corresponds to the value of $2a$.

Unlike supply in its arbitrary character, Q, the discharge from the lake is, of course, determined at any time solely by the value of h, or the level of its outlet at that time. Just what relation exists between Q and h is a matter of observation. Were the outlet a plain weir of length l, with H measuring the height above its crest, the relation would be $Q = 3.3\ l\ H^{\frac{3}{2}}$, while the form that has more generally been applied to the flow of rivers is $Q = C\ H^2$; where H again gives the level above an approximate zero of discharge, and C is a constant to be determined from the discharge observations. Either of these more general expressions, however, makes the solution of Equation I unnecessarily difficult, and in the small oscillation of a lake surface it is sufficient to take Q as made up of a constant element Q_1, the value of discharge at an assumed level, and a variable element $c\,h$, where h gives the surface at any time above or below this arbitrary level, and c is the average change in discharge there per foot of rise or fall. Where large discharges are dealt with, some 10 or 15 ft. above their zero level, there is little difference in a foot or so between the arc of the discharge curve and the straight line $c\,h$, and it would be hardly necessary to consider this, even in the flow of a river; while in the case of the flow out of the lake, where the accidental oscillation at its outlet may exceed the whole range of its true level, such a difference would seem to be an even more needless refinement. In what follows, therefore, the discharge at any time will be taken as $Q_1 + c\,h$, or a linear variation with the value of h at that time.

Substituting these values for S and Q in Equation I gives

$$[(S_1 - Q_1) + (a \sin t - c\,h)]\ d\,T = A\ d\,h.$$

And for the present neglecting the arbitrary constant $(S_1 - Q_1)$, which may always be made zero at any time by taking a suitable level, from which h is to be measured for that time, Equation I becomes

$$(a \sin t - c\,h)\ d\,T = A\ d\,h.$$

In this equation T is time in seconds, while t is an arc on a circle in which 2π is to represent one year. The relation between them is, therefore, given by the following proportion:

$$T : t = 86\,400 \times 365 \text{ (seconds in one year)} : 2\pi$$

or
$$T = \frac{86\,400 \times 365}{6.2832}\, t;$$

and
$$d\,T = \frac{86\,400 \times 365}{6.2832}\, dt.$$

At the same time, it is also convenient to change the unit of A from square feet to square miles, so that the equation becomes

$$(a \sin t - c\,h)\,\frac{86\,400 \times 365}{6.2832}\,dt = (5\,280)^2\,A\,dh$$

or
$$(a \sin t - c\,h)\,0.180036\,dt = A\,dh \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (II)$$

or
$$dh + \frac{0.18\,c}{A}\,h\,dt = \frac{0.18}{A}\,a \sin t\,dt$$

This is a well-known form, and is solved by multiplying through by

$$e^{\frac{0.18\,c}{A}t}$$

or
$$e^{\frac{0.18\,c}{A}t}\,dh + h\,\frac{0.18\,c}{A}\,e^{\frac{0.18\,c}{A}t}\,dt = \frac{0.18}{A}\,a \sin t\, e^{\frac{0.18\,c}{A}t}\,dt$$

Where the left-hand side of the equation is the complete differential of

$$h \times e^{\frac{0.18\,c}{A}t},$$

and the right contains only the single variable t and is easily integrated by parts as follows:

$$\frac{0.18}{A}\,a \int \sin t\, e^{\frac{0.18\,c}{A}t}\,dt = \frac{0.18}{A}\,a \int \sin t\, \frac{d\left(e^{\frac{0.18\,c}{A}t}\right)}{\frac{0.18\,c}{A}}$$

$$= \frac{0.18}{A}\,a \left[\frac{\sin t\, e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} - \frac{1}{\frac{0.18\,c}{A}} \int \cos t\, e^{\frac{0.18\,c}{A}t}\,dt\right]$$

And again:

$$\int \cos t\, e^{\frac{0.18\,c}{A}t}\,dt = \int \cos t\, \frac{d\left(e^{\frac{0.18\,c}{A}t}\right)}{\frac{0.18\,c}{A}}$$

$$= \frac{\cos t\, e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} + \frac{1}{\frac{0.18\,c}{A}} \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt.$$

Hence

$$\frac{0.18}{A}\, a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt = \frac{0.18}{A}\, a \left[\frac{\sin t\, e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} - \right.$$

$$\left. \frac{\cos t\, e^{\frac{0.18\,c}{A}t}}{\left(\frac{0.18\,c}{A}\right)^2} - \frac{1}{\left(\frac{0.18\,c}{A}\right)^2} \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt \right]$$

And solving for $\frac{0.18}{A}\, a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt$, gives

$$\frac{0.18}{A}\, a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt \left(1 + \frac{1}{\left(\frac{0.18\,c}{A}\right)^2}\right)$$

$$= \frac{0.18}{A}\, a\, \frac{1}{\left(\frac{0.18\,c}{A}\right)^2} \left[\frac{0.18\,c}{A} \sin t - \cos t\right] e^{\frac{0.18\,c}{A}t}$$

or, finally,

$$\frac{0.18}{A}\, a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, dt = \frac{0.18}{A}\, a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right] e^{\frac{0.18\,c}{A}t}$$

And the primitive of differential Equation II becomes

$$h\, e^{\frac{0.18\,c}{A}t} = \frac{0.18}{A}\, a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right] e^{\frac{0.18\,c}{A}t} \pm C \left\{ \begin{array}{l} \text{The constant of integration.} \end{array} \right.$$

or dividing by $e^{\frac{0.18\,c}{A}t}$

$$h = \frac{0.18}{A}\, a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right] \pm \frac{C}{e^{\frac{0.18\,c}{A}t}}$$

In this form it is plain that if the value of h is to be recurrent, or the surface movement is restricted to an annual oscillation, the term

$\frac{C}{e^{\frac{0.18\,c}{A}t}}$ must disappear, and the constant of integration must, therefore, be zero. In this case it will be called h_1 where uniformly

$$h_1 = \frac{0.18}{A}\,a\left[\frac{\frac{0.18\,c}{A}\sin t - \cos t}{1+\left(\frac{0.18\,c}{A}\right)^2}\right]$$

Here, also, for the excluded term $(S_1 - Q_1)$ to be zero, the mean supply, S_1, must be constant from year to year and equal to Q_1, a mean discharge, thus fixing the zero from which h is measured at a mean level. As, however, it is well known that the mean supply is not constant from year to year, and that it also passes through a more or less well-defined cycle covering a number of years, this term $(S_1 - Q_1)$ may be again written $(S_{11} - Q_{11}) + (a' \sin t' - c\,h')$, and this multiplied by $(0.18\,dt)$ equals $A\,dh'$, or a second additional element to the surface variation. Here, for all practical purposes, this longer cycle may be assumed as recurrent, with S_{11}, the mean supply, for the number of years equal to Q_{11}, the mean discharge, and the total variation of surface in that case made up of the sum of h_1 and h' measured from the mean level corresponding to Q_{11}.

In this longer cycle, however, the arc t' no longer corresponds to one year, but to a number of years n.

or while $$\frac{T}{t} = \frac{86\,400 \times 365}{2\pi}\,;\quad \frac{T}{t'} = \frac{n \times 86\,400 \times 365}{2\,\pi}$$

and $$t = n\,t',\ \text{and}\ dt = n\,dt'.$$

Hence the differential equation is

$$(a' \sin t' - ch')\ n \times 0.18\,dt' = A\,dh',$$

and its primitive is

$$h' = \frac{n \times 0.18}{A}\,a'\left[\frac{\frac{n \times 0.18\,c}{A}\sin t' - \cos t'}{1+\left(\frac{n \times 0.18\,c}{A}\right)^2}\right] \pm \frac{C}{e^{\frac{n \times 0.18\,c}{A}t'}}\,;$$

in which the term involving the constant of integration now disappears, by taking, as may in general be done, a recurrent variation of h'.

And as the additional oscillation has been taken through a number of years, so also an additional oscillation through a fraction of the year might be taken, and in a sum of these oscillations h may thus

be made to express all the variations of the surface for any such system of regular variations in the monthly and the annual supplies. It is, however, true that the supply in any actual case may be far from regular, and while $(Q_{11} + c\,h)$ always expresses the actual discharge (the difference between the curve and line as noted being insignificant) still $S_{11} + a \sin t + a' \sin t'$, etc., may not so express the actual supply, unless such altogether arbitrary values may be assigned to S_{11} from time to time as will make the expression practically fit any form of irregular variation. It is, therefore, desirable finally to solve Equation I in its most general form with $(S_{11} - Q_{11})$ simply an arbitrary constant, having, of course, alternating + and — values, which at any period of irregularity may be represented by $\triangle S$. In addition to h_1 and h' there is then a third element of variation h'', given by an additional term of the form

$$\frac{0.18}{A}\int \triangle S\, e^{\frac{0.18\,c}{A}t}\, dt$$

or

$$h'' = \frac{\triangle S}{c} \pm \frac{C}{e^{\frac{0.18\,c}{A}t}}$$

And here to determine the general form of the constant of integration C, it is necessary to consider how a lake would rise or fall with such a change in its supply as that of the arbitrary constant $\triangle S$.

Substituting then in Equation II there results

$$(\triangle S - c h'')\, 0.18\, d\,t = A\, d\,h''$$

or

$$d\,t = \frac{A}{0.18\,c}\,\frac{d\,h''}{\frac{\triangle S}{c} - h''}$$

or

$$t = -\frac{A}{0.18\,c}\log_e\left(\frac{\triangle S}{c} - h''\right) + C$$

And if t be taken as zero where h'' is zero and t_o represents any point of time in such an arbitrary period of t corresponding to the arbitrary value $\triangle S$,

then

$$C = \frac{A}{0.18\,c}\log_e\left(\frac{\triangle S}{c}\right)$$

and

$$t_o = \frac{A}{0.18\,c} \log_e \left[\frac{\frac{\triangle S}{c}}{\frac{\triangle S}{c} - h''} \right]$$

or

$$e^{\frac{0.18\,c}{A} t_o} = \frac{\frac{\triangle S}{c}}{\frac{\triangle S}{c} - h''}$$

or

$$h'' = \frac{\triangle S}{c} - \frac{\frac{\triangle S}{c}}{e^{\frac{0.18\,c}{A} t_o}}$$

From which it is seen that a general form for the constant of integration C is $-\frac{\triangle S}{c}$ which, of course, is zero in all the former cases where $\triangle S$ was assumed zero.

The complete equation may be now written as follows:

$$h = h_1 + h' + h'' = \frac{0.18}{A} a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right] + \frac{n \times 0.18}{A} a'$$

$$\times \left[\frac{\frac{n \times 0.18\,c}{A} \sin t' - \cos t'}{1 + \left(\frac{n \times 0.18\,c}{A}\right)^2} \right] + \frac{\triangle S}{c} \left[1 - \frac{1}{e^{\frac{0.18c}{A} t_o}} \right] \ldots\ldots \text{(III)}$$

In which t is a continuous angle on a scale in which 360° equal one year, and t_o is the length of a natural arc on the same scale, but taken only through the arbitrary periods in which values may be assigned to $\triangle S$; while again t' is a continuous angle on a scale in which 360° $= n$ years. In this form, in any lake, the function

$\frac{0.18}{A} a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right]$ may be computed once for all through a cycle, and its oscillation then gives at once the ratio of the oscillation of h_1 to a. Or, from the variation of surface in the mean monthly levels the corresponding variation in the supply, expressed by a, is gotten from this simpler proportion. In the same way for h', the var-

iation of supply from year to year expressed by a' may be brought into a like proportion with the variation in the mean annual levels. While finally for given periods of t_0 corresponding proportions may be made up that express immediately the $\triangle S$ for any irregularity.

The relation of the variation of level in any lake to its variable supply may be here left for the more general problem which includes not only this, but its resultant effect on a lake below, and so on through a chain of lakes; and not to needlessly extend the problem, it is desirable in this case to use the simplest possible form for h. It is plain that in so far as h_1 and h' are concerned, it is only necessary to use the term h_1, for what will be generally true of the short cycle will be equally true of the long, and all equations deduced from the first can be immediately converted into equations for the second by substituting in the place of (0.18) the value of (n 0.18) throughout, remembering, of course, that degrees in the angles t' are also to be taken as covering n times as many days as they do in the angles t; but the wholly different character of the term for h'' makes it necessary to exclude it altogether or to consider its effects down entirely as a question by itself.

This last has in fact been done, but it is all so much a matter of the particular case, that it will not be included in this study. Indeed, from the extreme where $\triangle S$ is assumed a permanent change in the mean supply, it is evident at once that in time any number of lakes would all come to about the same difference in level, represented by $\frac{\triangle S}{c}$; though it may be noted that none could absolutely reach this level until t_0 became infinite. And while in very long periods there are practically no differences in the effects of this function on lakes below, at the same time in very short periods it has practically no effect whatever on them, so that in the range of ordinary irregularities in supply, it may simply be taken as rapidly disappearing in its effects on the lower lakes, but exactly how rapidly would depend on the length of the irregularity. It is only perhaps in the event of a system of storage in an upper lake through one season to be let out at another that the subject would have any general interest and there its effects are best worked out as a special case.

The more general problem of the effects of variable supplies on the levels of a chain of lakes is therefore confined to the consideration of the recurrent yearly oscillations h_1 and its effects below; and the

following general system of notation is taken for the resulting surface movements in a chain of n lakes.

$$I\,h_1$$
$$I\,h_2 = II\,h_2$$
$$I\,h_3 = II\,h_3 = III\,h_3$$
$$I\,h_4 = II\,h_4 = III\,h_4 = IV\,h_4$$
$$I\,h_5 = II\,h_5 = III\,h_5 = IV\,h_5 = V\,h_5$$
etc., etc., etc., etc., etc.,
$$I\,h_n = II\,h_n = III\,h_n = IV\,h_n = V\,h_n \text{ etc. } [n]\,h_n.$$

Here the vertical lines represent the effects of the primary supply in the basin of each respective lake all the way down, and the horizontal represent all the effects that as a whole make up the surface movement of any lake in the system. In correspondence with this the areas of the respective lakes will be represented by A_1, A_2, etc., A_n, the "a" in the variations of supply $a \sin t$ by a_1, a_2, etc., a_n, and the change of discharge per foot of rise or fall in the outlets by C_1, C_2, etc., C_n.

Considering, now, the supply in the basin of the first lake and its effects down, there results:

$$I\,h_1 = a_1 \times \frac{0.18}{A_1} \times \frac{1}{1 + \left(\frac{0.18\,c_1}{A_1}\right)^2} \left[\frac{0.18\,c_1}{A_1} \sin t - \cos t\right]$$

and as the factors $\frac{0.18}{A}$ and $\frac{1}{1 + \left(\frac{0.18\,c}{A}\right)^2}$ will recur continually in these operations, it is convenient to represent them by the more simple terms K and V, writing them for their respective values of A and C, as K_1, K_2; V_1, V_2, etc.

With this substitution

$$I\,h_1 = a_1\,K_1\,V_1\,(K_1\,c_1 \sin t - \cos t)$$

and as for the second lake $c_1 \times h_1$ is the resulting variation of supply and $c_2 \times h_2$, the corresponding variation in its discharge, from equation II there results:

$$[a_1\,c_1\,K_1\,V_1\,(K_1\,c_1 \sin t - \cos t) - c_2\,h_2]\,0.18\,dt = A_2\,dh_2$$

or

$$dh_2 + \frac{0.18\,c_2}{A_2}\,h_2\,dt = \frac{0.18}{A_2}\,a_1\,c_1\,K_1\,V_1\,(K_1\,c_1 \sin t - \cos t)\,dt$$

or

$$h_2\,e^{K_2\,c_2\,t} = a_1\,c_1\,K_1\,K_2\,V_1 \int (K_1\,c_1 \sin t - \cos t)\,e^{K_2\,c_2\,t}\,dt.$$

And in general, following the process of the former integration by parts,

$$\int e^{K c t} \sin t \, dt = \frac{K c \sin t - \cos t}{1 + (K c)^2} \, e^{K c t}$$

and

$$\int e^{K c t} \cos t \, dt = \frac{K c \cos t + \sin t}{1 + (K c)^2} \, e^{K c t}$$

so that

$$h_2 e^{K_2 c_2 t} = a_1 c_1 K_1 K_2 V_1 V_2 \, | K_1 c_1 (K_2 c_2 \sin t - \cos t) - (K_2 c_2 \cos t + \sin t)] \, e^{K_2 c_2 t}$$

or

$$I h_2 = a_1 c_1 K_1 K_2 V_1 V_2 \, [(K_2 c_2 K_1 c_1 - 1) \sin t - (K_2 c_2 + K_1 c_1) \cos t].$$

And again for the third lake with $c_2 \times h_2$ supply, and $c_3 \times h_3$ discharge, $h_3 e^{K_3 c_3 t} = a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 \int [(K_2 c_2 K_1 c_1 - 1) \sin t - (K_2 c_2 + K_1 c_1) \cos t] \, e^{K_3 c_3 t} dt = a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 V_3 [(K_2 c_2 K_1 c_1 - 1)(K_3 c_3 \sin t - \cos t) - (K_2 c_2 + K_1 c_1)(K_3 c_3 \cos t + \sin t)] \, e^{K_3 c_3 t}$

or $I h_3 =$

$$a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 V_3 \Big[[K_3 c_3 (K_2 c_2 K_1 c_1 - 1) - (K_2 c_2 + K_1 c_1)] \sin t - [K_3 c_3 (K_2 c_2 + K_1 c_1) + (K_2 c_2 K_1 c_1 - 1)] \cos t \Big]$$

and, in general, calling*

$$I h_n = a_1 \{c_1\}^{n-1} \{K_1\}^n \{V_1\}^n [I \alpha_n \sin t + I \beta_n \cos t] \ldots\ldots (IV)$$

in which α and β are formed as follows:

	$I \alpha =$	$I \beta =$
$I h_1 =$	$(K_1 c_1)$	$= -1$
$I h_2 =$	$(K_2 c_2 K_1 c_1 - 1)$	$= -(K_2 c_2 + K_1 c_1)$
$I h_3 =$	$K_3 c_3 (K_2 c_2 K_1 c_1 - 1) - (K_2 c_2 + K_1 c_1)$.	$= -[K_3 c_3 (K_2 c_2 + K_1 c_1) + (K_2 c_2 K_1 c_1 - 1)]$

and finally

$$I h_n = K_n c_n \alpha_{n-1} + \beta_{n-1} \ldots\ldots = K_n c_n \beta_{n-1} - \alpha_{n-1}.$$

In the case of $II h_2$ begin with

$$II h_2 = a_2 K_2 V_2 [K_2 c_2 \sin t - \cos t]$$

and in the same way finally in the n^{th} lake there results

$$II h_n = a_2 \{c_2\}^{n-1} \{K_2^n\} \{V_2^n\} [II \alpha_n \sin t + II \beta_n \cos t]$$

and so on from lake to lake.

From these equations the effect of one lake upon another, and the total surface movements in any lake, are easily computed. Thus, to

* In this paper the braces $\{ \}$ are used exclusively as a "factorial sign."

illustrate, the case of a chain of five lakes may be taken, each with a surface area of 10 000 square miles, with a 100 000 cu. ft. per second variation in the supply, entering each lake as a simultaneous oscillation, and with outlet capacities downward of, respectively, 10, 15, 20, 25 and 30 thousands of cubic feet per second per foot of rise or fall. Then

$$A_1 = A_2 = A_3 = A_4 = A_5 = 10\,000$$

$$a_1 = a_2 = a_3 = a_4 = a_5 = 50\,000$$

$C_1 = 10\,000$; $C_2 = 15\,000$; $C_3 = 20\,000$; $C_4 = 25\,000$; $C_5 = 30\,000$.

Hence

$K C_1 = 0.18$; $K C_2 = 0.27$; $K C_3 = 0.36$; $K C_4 = 0.45$; $K C_5 = 0.54$

and

$I\alpha_1 = +0.18$

0.27

126

036

$+0.0486$

-1.0000

$I\alpha_2 = -0.9514$

0.36

57084

28542

-0.3425

-0.4500

$I\alpha_3 = -0.7925$

0.45

39625

31700

-0.3566

$+0.7894$

$I\alpha_4 = +0.4328$

0.54

17312

21640

$+0.2337$

$+1.1477$

$I\alpha_5 = +1.3814$

$I\beta_1 = -1.0$

0.27

-0.27

-0.18

$I\beta_2 = -0.45$

0.36

270

135

-0.1620

$+0.9514$

$I\beta_3 = +0.7894$

0.45

39470

31576

$+0.35523$

$+0.7925$

$I\beta_4 = +1.1477$

0.54

45908

57385

$+0.6198$

-0.4328

$I\beta_5 = +0.1870$

and so for $II\,\alpha_2$, $II\,\beta_2$, to $II\,\alpha_5$, $II\,\beta_5$, etc., giving:

α_1	α_2	α_3	α_4	α_5	
+	−	−	+	+	
0.1800	0.9514	0.7925	0.4328	1.3814	*I*
	+	−	−	+	
	0.2700	0.9028	1.0363	0.0597	*II*
		+	−	−	
		0.3600	0.8380	1.2625	*III*
			+	−	
			0.4500	0.7570	*IV*
				+	
				0.5400	*V*

β_1	β_2	β_3	β_4	β_5	
−	−	+	+	+	
1.0000	0.4500	0.7894	1.1477	0.1870	*I*
	−	−	+	+	
	1.0000	0.6300	0.6193	1.3707	*II*
		−	−	+	
		1.0000	0.8100	0.4006	*III*
			−	−	
			1.0000	0.9900	*IV*
				−	
				1.0000	*V*

Again, the following are the logs of the general factor $\alpha \left\{c\right\}^{n-1} \left\{K\ V\right\}^n$ which are easily made up by repeated additions:

$\left\{\ \right\}_1$	$\left\{\ \right\}_2$	$\left\{\ \right\}_3$	$\left\{\ \right\}_4$	$\left\{\ \right\}_5$	
−	−	−	−	−	
1.940394	1.165108	2.543547	2.019764	3.561849	*I*
	−	−	−	−	
	1.923683	1.302122	2.778940	2.321024	*II*
		−	−	−	
		1.901318	1.377535	2.919620	*III*
			−	−	
			1.874157	1 416242	*IV*
				−	
				1.843114	*V*

and multiplying here the respective values of α and β by the corresponding general factors and representing the products by $I\left\{\alpha_1\right\}$, $I\left\{\beta_1\right\}$, &c., there results finally:

$\left\{\alpha_1\right\}$	$\left\{\alpha_2\right\}$	$\left\{\alpha_3\right\}$	$\left\{\alpha_4\right\}$	$\left\{\alpha_5\right\}$	
+	−	−	+	+	
0.15692	0.13915	0.02770	0.00453	0.00504	*I*
	+	−	−	+	
	0.22649	0.18101	0.06229	0 00125	*II*
		+	−	−	
		0.28683	0 19988	0 10492	*III*
			+	−	
			0.33680	0.19740	*IV*
				+	
				0.37628	*V*

$\{\beta_1\}$	$\{\beta_2\}$	$\{\beta_3\}$	$\{\beta_4\}$	$\{\beta_5\}$	
—	—	+	+	+	
0.87176	0.06581	0.02760	0.01201	0.00068	*I*
	—	—	+	+	
	0.83885	0.12632	0.03723	0.02871	*II*
		—	—	+	
		0.79674	0.19321	0.03329	*III*
			—	—	
			0.74844	0.25815	*IV*
				—	
				0.69681	*V*

Taking now t at intervals of 30° corresponding with the twelfth of the year and practically with the monthly means:

t	sin t	cos t
0°	0	+1
30°	+0.5	+0.866
60°	+0.866	+0.5
90°	+1	0
120°	+0.866	−0.5
150°	−0.5	−0.866
180°	0	−1

And with these values, from the equation $h = \{\alpha\} \sin t + \{\beta\} \cos t$, the whole cycle of each of these variations is readily made up as follows:

$t =$	0°	30°	60°	90°	120°	150°	180°
$I \{\alpha_1\} \sin t =$	+0.0000	+0.0785	−0.1359	−0.1569	−0.1359	+0.0785	+0.0000
$I \{\beta_1\} \cos t =$	−0.8718	−0.7550	−0.4359	−0.0000	−0.4359	+0.7550	+0.8718
$I h_1 =$	−0.8718	−0.6765	−0.3000	+0.1569	+0.5718	+0.8335	+0.8718

and with signs reversed

$t =$	+ 210°	+ 240°	− 270°	− 300°	− 330°	− 360°

completing the cycle.

These values for the half cycle are here given in series from the primary supply on, to its effect in the last lake.

	0°	30°	60°	90°	120°	150°	180°
$I h_1$	−0.8718	−0.6765	−0.3000	+0.1569	+0.5718	+0.8335	+0.8718
$I h_2$	−0.0658	−0.1266	−0.1534	−0.1391	−0.0876	−0.0126	+0.0658
$I h_3$	+0.0276	+0.0101	−0.0102	−0.0277	−0.0378	−0.0377	−0.0276
$I h_4$	+0.0120	+0.0127	+0.0099	+0.0045	−0.0021	−0.0081	−0.0120
$I h_5$	+0.0007	+0.0031	+0.0047	+0.0050	+0.0041	+0.0019	−0.0007
$II h_2$	−0.8388	−0.6133	−0.2233	+0.2265	+0.6155	+0.8397	+0.8388
$II h_3$	−0.1263	−0.1999	−0.2200	−0.1810	−0.0936	+0.0189	+0.1263
$II h_4$	+0.0372	−0.0011	−0.0353	−0.0622	−0.0725	−0.0633	−0.0372
$II h_5$	+0.0287	+0.0255	+0.0155	+0.0012	−0.0133	−0.0243	−0.0287
$III h_3$	−0.7967	−0.5466	−0.1500	+0.2868	+0.6468	+0.8334	+0.7967
$III h_4$	−0.1932	−0.2672	−0.2697	−0.1999	−0.0765	+0.0674	+0.1932
$III h_5$	+0.0333	−0.0237	−0.0743	−0.1049	−0.1075	−0.0813	−0.0333
$IV h_4$	−0.7484	−0.4798	−0.0825	+0.3368	+0.6659	−0.8166	+0.7484
$IV h_5$	−0.2581	−0.3223	−0.3000	−0.1974	−0.0418	−0.1249	+0.2581
$V h_5$	−0.6968	−0.4154	−0.0225	+0.3763	+0.6743	+0.7916	+0.6968

And these values summed algebraically for their respective lakes would give in each case the total of the surface movement there, but it is easier to sum at once the coefficients of sin t and cos t and calculate directly the cycles of these resultant oscillations.

Giving the following values:

		0°	30°	60°	90°	120°	150°	180°
I	h_1	−0.8718	−0.6765	−0.3000	+0.1569	+0.5718	+0.8335	+0.8718
I to *II*	h_2	−0.9047	−0.7398	−0.3767	+0.0873	+0.5279	+0.8272	+0.9047
I to *III*	h_3	−0.8955	−0.7364	−0.3801	+0.0781	+0.5153	+0.8146	+0.8955
I to *IV*	h_4	−0.8924	−0.7332	−0.3777	+0.0792	+0.5147	+0.8124	+0.8924
I to *V*	h_5	−0.8922	−0.7326	−0.3766	+0.0802	+0.5156	+0.8128	+0.8922

These values are shown graphically in Fig. 1, and while they show the general form of the oscillations, it is very evident that they do not give, with any degree of accuracy, the extremes. It is seen, of course, that while $a \sin t$, or the supply, is in all cases at its maximum at 90°, its corresponding rise reaches its maximum in the first lake somewhere between the 150° and 180° periods, and that this again as a supply is once more retarded in the rise of the second lake, and so on from lake to lake; but the periods and the values of these maxima require additional calculations.

The value of t at these points of maxima or minima is, of course, determined by the condition $\frac{dh}{dt} = 0$, and in the general form of the equation

$$h_n = a \left\{c\right\}^{n-1} \left\{K\right\}^{n} \left\{V\right\}^{n} [\alpha_n \sin t + \beta_n \cos t]$$

$$\frac{dh_n}{dt} = 0, \text{ gives } \alpha_n \cos t = \beta_n \sin t, \text{ or } \tan t = \frac{\alpha}{\beta}.$$

The two angles in the cycle differing by 180°, that correspond to this value of the tan t, are, of course, the points of maximum and minimum respectively ; and by substituting either in the equation for h the + or − extreme is given. It is, however, enough simply to consider the maximum, as before only the half cycle was calculated, every value of h having, of course, a corresponding value with an opposite sign after a period of 180°.

As the effects of a primary supply are traced down, the retardation throws the maximum into later and later quadrants, and these must be followed by the successive change of signs in the term $\frac{\alpha}{\beta}$.

Thus, in the case of *I*, the original supply, $a \sin t$, has its maximum as noted at 90°, while for *I* h_1, $\frac{\alpha_1}{\beta_1}$ is minus, and is therefore in the

second quadrant, and its angle 10° 12′ gives the absolute position as 180° 00′ — 10° 12′ = 169° 48′, and substituting this angle for t gives the value of 0.8858 for the maximum of $I\,h_1$. Again for $I\,h_2$, $\frac{\alpha_2}{\beta_2}$ is plus, and is thus in the third quadrant, and its angle 64° 41′ gives its position as 180° 00′ + 64° 41′ = 244° 41′, and its value 0.1539. In the same way $\frac{\alpha_3}{\beta_3}$ is minus and lies in the fourth quadrant, and its angle 45° 07′ places it at 360° 00′ — 45° 07′ = 314° 53′ with the maximum value 0.0391. While finally $\frac{\alpha_4}{\beta_4}$ and $\frac{\alpha_5}{\beta_5}$ are both plus and lie in the fifth quadrant at the points respectively 360° 00′ — 20° 40′ = 380° 40′ and 360° 00′ + 82° 18′ = 442° 18′, and maxima of 0.0128 and 0.0051.

The positions of these maxima and their values through this whole series of oscillations are tabulated as follows:

Angle t of maximum.

h_1	h_2	h_3	h_4	h_5	
169° 48′	244° 41′	314° 53′	380° 40′	442° 18′	*I*
	164° 53′	235° 05′	300° 52′	362° 30′	*II*
		160° 12′	225° 58′	287° 36′	*III*
			155° 46′	217° 24′	*IV*
				151° 38′	V

Value of maximum h.

h_1	h_2	h_3	h_4	h_5	
0.8858	0.1539	0.0391	0.0128	0.0051	*I*
	0.8689	0.2207	0.0726	0.0287	*II*
		0.8468	0.2780	0.1101	*III*
			0.8207	0.3250	*IV*
				0.7919	V

From these values of t it is seen at once that the difference between the maxima of $I\,h_1$ and $I\,h_2$, or 244° 41′ — 169° 48′ = 74° 53′ is also the difference between 164° 53′ and 90°, or between the maximum of supply, $a \sin t$, and its rise, $II\,h_2$. And, again, 70° 12′ is the general difference between h_2 and h_3; 65° 46′ + between h_3 and h_4, and 61° 38′ between h_4 and h_5; which are also in each case the difference between 90°, the maxima of the respective supplies, $a \sin t$, and their corresponding oscillations in the first lake. It is then plain that the retardation in any lake is a constant dependent upon the physical proper-

ties of that lake, and in no way upon the magnitude of the oscillations passing through it. And just as it has been seen that any oscillation passing from a given lake to the next is retarded by a fixed time ; so it may be seen from the corresponding maximum values of h that they are also reduced by a fixed percentage.

It is time therefore to turn from the series of equations which have just been considered, to determine the formulas for this retardation and reduction from lake to lake ; for it is very evident that they are a much more general expression for the laws of lake movements than equations which simply give direct values for assumed times and supplies.

First, for the reduction ratio or percentage, which will be called p ; noting that it is enough to express it in the case of I from h_1 to h_n, for it is also the same from lake to lake in the case of II, III, etc. ; and omitting, therefore, the I in the general equation, and representing by h' and t' respectively the maximum rise and the time corresponding to it, there results:

$$h'_n = a \left\{c_1\right\}^{n-1} \left\{K_1\right\}^n \left\{V_1\right\}^n [\alpha_n \sin t' + \beta_n \cos t'] \quad \ldots\ldots\ldots\ldots \quad (1)$$

and

$$\frac{\alpha_n}{\beta_n} = \frac{\sin t'}{\cos t'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

or from (2)

$$\sin t' = \frac{\alpha_n}{\sqrt{\alpha_n^2 + \beta_n^2}} \text{ and } \cos t' = \frac{\beta_n}{\sqrt{\alpha_n^2 + \beta_n^2}}$$

and eliminating t' between equations (1) and (2)

$$h'_n = a \left\{c_1\right\}^{n-1} \left\{K_1\right\}^n \left\{V_1\right\}^n \sqrt{\alpha_n^2 + \beta_n^2}.$$

Again from the law of formation in the general values of α and β, there results:

$\alpha_1 = K_1 c_1$ or $\alpha_1^2 = \overline{K_1 c_1}^2$. $\beta_1 = -1$; $\beta_1^2 = 1$. Hence, $\alpha_1^2 + \beta_1^2 = 1 + \overline{K_1 c_1}^2$

And again:

$$\alpha_2 = K_2 c_2 \alpha_1 + \beta_1 \text{ or } \alpha_2^2 = \overline{K_2 c_2}^2 \alpha_1 + 2 K_2 c_2 \alpha^2_1 \beta_1 + \beta_1^2$$

$$\beta_2 = K_2 c_2 \beta_1 - \alpha_1 \text{ or } \beta_2^2 = \overline{K_2 c_2}^2 \beta_1^2 - 2 K_2 c_2 \alpha_1 \beta_1 + \alpha_1^2$$

$$\text{Hence, } \alpha_2^2 + \beta_2^2 = \overline{K_2 c_2}^2 \left(\alpha_1^2 + \beta_1^2\right) + \alpha_1^2 + \beta_1^2$$

or

$$\alpha_2^2+\beta_2^2=\left(\alpha_1^2+\beta_1^2\right)\left(1+\overline{K_2c_2}^2\right)=\left(1+\overline{K_1c_1}^2\right)\left(1+\overline{K_2c_2}^2\right)$$

and so on, until in general

$$\alpha_n^2+\beta_n^2=(1+\overline{K_1c_1}^2)\ (1+\overline{K_2c_2}^2)\ (1+\overline{K_3c_3}^2)\ \ \ldots\ (1+\overline{K_nc_n}^2)=\left\{1+\overline{K_1c_1}^2\right\}^n$$

from which finally there is obtained—

$$\frac{h'_n}{h'_{n-1}}=\frac{a\left\{c_1\right\}^{n-1}\left\{K_1\right\}^n\left\{V_1\right\}^n\sqrt{\left\{1+\overline{K_1c_1}^2\right\}^n}}{a\left\{c_1\right\}^{n-2}\left\{K_1\right\}^{n-1}\left\{V_1\right\}^{n-1}\sqrt{\left\{1+\overline{K_1c_1}^2\right\}^{n-1}}}=p_n$$

Or

$$p_n=c_{n-1}\,K_n\,V_n\sqrt{1+\overline{K_nc_n}^2}$$

And writing now for K_n and V_n their respective values:

$$K_n=\frac{0.18}{A_n}$$

$$V_n=\frac{1}{1+\left(\frac{0.18\,c_n}{A_n}\right)^2}$$

$$p_n=\frac{0.18\,c_{n-1}}{A_n}\,\frac{\sqrt{1+\left(\frac{0.18\,c_n}{A_n}\right)^2}}{1+\left(\frac{0.18\,c_n}{A_n}\right)^2}=\frac{\frac{0.18\,c_{n-1}}{A_n}}{\sqrt{1+\left(\frac{0.18c_n}{A_n}\right)^2}}\ \ldots\ldots(V)$$

A value depending simply upon the area of the n^{th} lake and the discharging capacity of its inlet and its outlet.

Again, taking also the case *I* for the retardation, which will be called *R;* and representing by t'_{n-1} and t'_n, the time of the maxima in the $n-1$ and the n^{th} lake, respectively, there results:

$$\sin R_n=\sin\left(t'_n-t'_{n-1}\right)=\sin t'_n\cos t'_{n-1}-\cos t'_n\sin t'_{n-1}$$

and as before

$$\sin t_n{}^1=\frac{\alpha_n}{\sqrt{\alpha_n{}^2+\beta_n{}^2}}=\frac{\alpha_n}{\sqrt{\left\{1+\overline{K_1c_1}^2\right\}^n}}\text{ and }\cos t'_n=\frac{\beta_n}{\sqrt{\alpha_n{}^2+\beta_n{}^2}}=\frac{\beta_n}{\sqrt{\left\{1+\overline{K_1c_1}^2\right\}^n}}$$

Hence,

$$\sin R_n=\frac{\alpha_n\,\beta_{n-1}}{\left\{1+\overline{K_1c_1}^2\right\}^{n-1}\sqrt{1+\overline{K_nc_n}^2}}-\frac{\alpha_{n-1}\,\beta_n}{\left\{1+\overline{K^1c_1}^2\right\}^{n-1}\sqrt{1+\overline{K_nc_n}^2}}$$

and as $$\alpha_n\,\beta_{n-1}=(K_n\,c_n\,\alpha_{n-1}+\beta_{n-1})\,\beta_{n-1}$$

and $$\alpha_{n-1}\,\beta_n = (K_n\,c_n\,\beta_{n-1} - \alpha_{n-1})\,\alpha_{n-1}$$

$$\alpha_n\,\beta_{n-1} - \alpha_{n-1}\,\beta_n = \alpha^2{}_{n-1} + \beta^2{}_{n-1} = \left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}$$

therefore,

$$\sin R_n = \frac{\left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}}{\left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}\sqrt{1 + \overline{K_n\,c_n}^2}} = \frac{1}{\sqrt{1 + \overline{K_n\,c_n}^2}} =$$

$$\frac{1}{\sqrt{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (VI)$$

From equation (VI) it is seen that in passing through a given lake, oscillations of supply are retarded by this period, which depends alone upon the area and the outlet of that lake; but on comparing it with equation (V) it is evident at once that p_n is not a reduction of the same general character. It gives the ratio $\frac{h'_n}{h'_{n-1}}$, and is a most convenient function for calculating the effects of a given oscillation in any lake upon all the lakes that follow it; for at any point $h'_{n-1} \times p_n = h'_n$; but with exactly the same oscillation of supply in the n^{th} lake, and, in consequence, the same value of h'_n, p_n may have any number of values, as the oscillation from above which gives this supply has different outlets.

Thus, finally, there may be considered the reduction effected by a given lake on the variations of supply; or a reduction of the same general character as the retardation; and calling $\triangle S'_n$ and $\triangle Q'_n$ the maxima for the oscillations of supply and discharge respectively in the n^{th} lake, this absolute percentage of reduction, which will be called P, is given by the ratio $\frac{\triangle Q'_n}{\triangle S'_n}$.

This is easily determined from equation (V),

for $$h'_n c_n = \triangle Q'_n \text{ and } h'_{n-1}\,c_{n-1} = \triangle S'_n$$

or $$\frac{\triangle Q'_n}{\triangle S'_n} = \frac{h'_n}{h'_{n-1}} \times \frac{c_n}{c_{n-1}}$$

and as $$p_n = \frac{h'_n}{h'_{n-1}} = \frac{\frac{0.18\,c_{n-1}}{A_n}}{\sqrt{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}}$$

there results

$$p_n = \frac{c_n}{c_{n-1}} \times \frac{\frac{0.18\ c_{n-1}}{A_n}}{\sqrt{1 + \left(\frac{0.18\ c_n}{A_n}\right)^2}} = \frac{\frac{0.18\ c_n}{A_n}}{\sqrt{1 + \left(\frac{0.18\ c_n}{A_n}\right)^2}} \quad \ldots\ldots\ldots (VII)$$

Thus in equations (*VI*) and (*VII*) may be seen finally, from the area and the outlet of the lake itself, its whole effect upon any oscillations of supply. It simply retards it by this given time, and reduces it to this given percentage.

Indeed, these last equations might have been originally determined simply from the consideration of the general relations between an oscillation of supply, $a \sin t$, and its corresponding oscillation of level.

Or as before

$$h = a \frac{\frac{0.18}{A}}{1 + \left(\frac{0.18\ c}{A}\right)^2} \left[\frac{0.18\ c}{A} \sin t - \cos t\right]$$

and again taking t' as the angle corresponding to the maximum h' there results from $\frac{dh}{dt} = 0$

$$-\frac{0.18\ c}{A} \cos t' = \sin t'; \text{ and } \sin t' = \frac{\frac{0.18\ c}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)^2}}$$

and

$$h \sin t' = a \frac{\frac{0.18}{A}}{1 + \left(\frac{0.18\ c}{A}\right)^2} \left[\frac{0.18\ c}{A} (\sin t \sin t' + \cos t \cos t')\right]$$

or

$$h \times \frac{\frac{0.18\ c}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)^2}} = a \times \frac{\frac{0.18}{A}}{1 + \left(\frac{0.18\ c}{A}\right)^2} \times \frac{0.18\ c}{A} \cos (t - t')$$

and finally

$$h = \frac{\frac{0.18}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)}} a \sin (90^\circ + t - t') \quad \ldots\ldots\ldots\ldots (VIII)$$

Which, simply reduced by the factor $\frac{\frac{0.18}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)^2}}$, is alto-

gether a similar oscillation to $a \sin t$, only shifted to a new origin of time, with a zero of 90° before the maximum t', or zero where $t' - 90° = t$, as the origin in the original, $a \sin t$, was zero 90° before its maximum; and as this is true of any oscillation in any lake, it is just as applicable to the n^{th} oscillation as the first; and the formulas for reduction and retardation determined from it apply to all.

From equation (*VIII*) it is very evident that the maximum of level is given in all cases by the equation

$$h' = \frac{\frac{0.18}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)^2}}\ a,$$

where a represents the maximum of any supply oscillation from any source.

And writing f for this function $\dfrac{\frac{0.18}{A}}{\sqrt{1 + \left(\frac{0.18\ c}{A}\right)^2}}$,

and for a given lake in a series f_n, there results $\dfrac{h'_n}{f_n} = a_n$, by which, from the observed maximum of rise h'_n in any case, the corresponding supply may be at once determined; while, again, this is easily carried down through any number of lakes by their successive values of the retardation and the reduction ratio.

An analysis of the mean oscillations of the Great Lakes is here given to illustrate the application of the above formulas to the study of such data.

In this case the following data are given :

	A.	*C.*	*Log f.*	*Log p.*	*t'.*	*R.*
I. Superior	31 800	17 000	$\bar{6}.7505$	.....	174° 30′	84 30′
II. Michigan-Huron	45 600	26 000	$\bar{6}.5944$	$\bar{2}.8245$	174° 08′	84° 08′
III. St. Clair	495	26 000	$\bar{5}.5826$	$\bar{1}.9976$	96° 02′	6° 02′
IV. Erie	10 000	30 000	$\bar{5}.1995$	$\bar{1}.6147$	151° 38′	61° 38′
V. Ontario	7 450	34 000	$\bar{5}.2711$	$\bar{1}.7482$	140° 36′	50° 36′

The means of twenty-five years, from 1871 to 1895, for Superior are as follows:

Observed oscillations.				Computed trial oscillations.		
				t	Ih_1	t
September.	+0.47	+0.47	September...	174°	+0.500	174°
				− 30°		+ 30°
August.....	+0.45	+0.44	October.....	144°	+0.433	204
				− 30°		+ 30°
July.....	+0.37	+0.29	November...	114°	+0.250	234°
				− 30°		+ 30°
June.....	+0.16	+0.01	December...	84°	0.000	264°
				− 30°		− 30°
May.....	−0.15	−0.25	January.....	54°	−0.250	294°
				− 30°		+ 30°
April.....	−0.45	−0.40	February....	24°	−0.433	324
				− 30°		+ 30°
March.....	−0.46	−0.46	March.....	− 06°	−0.500	354°

By plotting and comparing this trial oscillation with the observed one, it is seen that the zero of its t corresponds closely with the mean March period, and that its range fits the whole series of observed values about as well as possible.

The value of the maximum of Ih_1 is therefore taken as 0.50, and the oscillation of its supply is therefore—

$$\begin{aligned} \log \\ 0.50 &= \overline{1}.6990 \\ f_1 &= 6.7505 \\ a_1 &= 4.9495 \quad = 83\ 480. \end{aligned} \quad \text{or } IS = 83\ 480 \sin t$$

The effects of this upon all the lakes below is readily carried down as follows:

MAXIMUM EFFECTS.

			log.
It'_1	$= 174°\ 30'$	Ih'_1	$= \overline{1}.6990 = 0.500$
$+ R_2$	$84°\ 08'$	$- \log p_2$	2.8245
It'_2	$= 258°\ 38'$	Ih'_2	$= \overline{2}.5235 = 0.033$
$+ R_3$	$6°\ 02'$	$+ \log p_3$	1.9976
It'_3	$= 264°\ 40'$	Ih'_3	$= \overline{2}.5211 = 0.033$
$+ R_4$	$61°\ 38'$	$- \log p_4$	1.6147
It'_4	$= 326°\ 18'$	Ih'_4	$= \overline{2}.1358 = 0.014$
$+ R_5$	$50°\ 36'$	$+ \log p_5$	1.7482
It'_5	$= 376°\ 54'$	Ih'_5	$= \overline{3}.8840 = 0.008$

Again, from these times and values of the maxima, the complete oscillations are as follows:

Michigan-Huron.		St. Clair.		Erie.		Ontario.	
t.	Ih_2.	t.	Ih_3.	t.	Ih_4.	t.	Ih_5.
259°	+0.033	265°	+0.033	326°	+0.014	377°	+0.008
— 30°		— 30°		— 30°		— 30°	
229°	+0.029	235°	+0.029	296°	+0.012	347°	+0.007
— 30°		— 30°		— 30°		— 30°	
199°	+0.017	205°	+0.017	266°	+0.007	317°	+0.004
— 30°		— 30°		— 30°		— 30°	
169°	+0.000	175°	0.000	236°	0.000	287°	0.000
— 30°		— 30°		— 30°		— 30°	
139°	—0.017	145°	—0.017	206°	—0.007	257°	—0.004
— 30°		— 30°		— 30°		— 30°	
109°	—0.029	115°	—0.029	176°	—0.012	227°	—0.007
— 30°		— 30°		— 30°		— 30°	
79°	—0.033	— 85°	—0.033	146°	—0.014	197°	—0.008

Again, from the monthly means of the Michigan-Huron data, the effect there of the oscillation from Superior is first subtracted, and the result gives the oscillation of Michigan-Huron from the supply of its own water-shed. This is called the local observed oscillation, and with its corresponding computed trial oscillation is given as follows:

Local observed oscillation.				Computed trial oscillation.		
				t.	IIh_2	t.
July.....	+0.52	+0.52	July.....	174°	+0.520	174°
June.....	+0.49	+0.46	August.....	144°	+0.450	204°
May	+0.23	+0.22	September.	114°	+0.260	234°
April	—0.03	—0.03	October....	84°	0.000	264°
March.....	—0.27	—0.29	November..	54°	—0.260	294°
February....	—0.42	—0.53	December..	24°	—0.450	324°
January.....	—0.48	—0.48	January...	— 06°	—0.520	354°

Plotting and comparing these oscillations, it is seen that the maximum of 0.52 is best fitted to the whole series, from which—

$$\begin{aligned} &\log \\ 0.52 &= \overline{1}.7160 \\ f_2 &= \overline{6}.5944 \\ a_2 &= \overline{5}.1216 = 129\ 470 \end{aligned}$$

or $II\,S = 129\ 470 \sin t$.

The zero of t, however, must in this case be put some 4° later than the mid-January period; and to bring here the computed values of this oscillation and its effects below to the same time scale as Superior, 56°

must in all cases be subtracted from its values of t. This, however, may be done at once on the retardations, and then the Michigan-Huron effects on the Superior time scale are as follows:

MAXIMUM EFFECTS.

Michigan-Huron time.	$II\,t'_2$	$= 174°\ 08'$		
		$- \ 56°$		
Superior time.	$II\,t'_3$	$= 118°\ 08'$	$II\,h'_2$	log $\bar{1}.7160 = 0.520$
	$+ R_3$	$6°\ 02'$	$+ \log p_3$	$\bar{1}.9976$
	$II\,t'_3$	$= 124°\ 10'$	$II\,h'_3$	$= \bar{1}.7136 = 0.517$
	$+ R_4$	$61°\ 38'$	$+ \log p_4$	$\bar{1}.6147$
	$II\,t'_4$	$= 185°\ 48'$	$II\,h'_4$	$= \bar{1}.3283 = 0.213$
	$+ R_5$	$50°\ 36'$	$+ \log p_5$	$\bar{1}.7482$
	$II\,t'_5$	$= 236°\ 24'$	$II\,h'_5$	$= \bar{1}.0765 = 0.119$

And as before:

COMPLETE OSCILLATIONS.

ST. CLAIR.		ERIE.		ONTARIO.	
t.	$II\,h_3$.	t.	$II\,h_4$.	t.	$II\,h_5$.
124°	+0.517	186°	+0.213	236°	+0.119
94°	+0.448	156°	+0.184	206°	+0.103
64°	+0.259	126°	+0.107	176°	+0.060
34°	0.000	96°	0.000	146°	0.000
04°	—0.259	66°	—0.107	116°	—0.060
— 26°	—0.448	36°	—0.184	86°	—0.103
— 56°	—0.517	06°	—0.213	56°	—0.119

In the case of St. Clair there is lacking the observed oscillation to start with, but this may be closely approached by assuming, for its local supply a value of a_3 corresponding to its water-shed, and summing the effects of this with the effects from above.

In this case a_3 was finally taken as most probably about 8 000 with the same period as the Michigan-Huron supply, giving $S = 8\,000 \sin t$, where the t is reckoned on the Michigan-Huron scale.

Here then

$$III\,h'_3 = a_3 f_3 \begin{cases} 8\,000 & = 3.9031 \\ f_3 & = \bar{5}.5826 \\ III\,h'_3 & = \bar{1}.4857 = 0.306; \end{cases}$$

and the following are the

MAXIMUM EFFECTS:

Michigan-Huron time.	$III\,t'_3$	= 96° 02′		log
		56°		
Superior time.	$III\,t'_3$	= 40° 02′	$III\,h'_3$	= $\overline{1}.4857$ = 0.306
	$+R_4$	61° 38′	= log p_4	$\overline{1}.6147$
	$III\,t'_4$	= 101° 40′	$III\,h'_4$	= $\overline{1}.1004$ = 0.126
	$+R_5$	50° 36′	+ log p_5	$\overline{1}.7482$
	$III\,t'_5$	= 152° 16′	$III\,h'_5$	= $\overline{2}.8486$ = 0.070

And as before

COMPLETE OSCILLATIONS:

ST. CLAIR.		ERIE.		ONTARIO.	
t.	$III\,h_3$.	t.	$III\,h_4$.	t.	$III\,h_5$.
40°	+0.306	102°	+0.126	152°	+0.070
70°	+0.265	132°	+0.109	122°	+0.061
100°	+0.153	162°	+0.063	92°	+0.035
130°	0.000	192°	0.000	62°	0.000
160°	−0.153	222°	−0.063	32°	−0.035
190°	−0.265	252°	−0.109	02°	−0.061
220°	−0.306	282°	−0.126	−28°	−0.070

In this case in the place of the computed trial oscillation, the complete oscillation of St. Clair is taken to add to the effects from above.

For Erie, again subtracting the effects of the three lakes above from its mean levels, the following is left for the local observed oscillation and the computed oscillation corresponding to it.

Local observed oscillation.				Computed Trial Oscillation.		
				t	$IV\,h_4$	t
June	+0.545	+0.545	June.....	152	+0.500	152
May.....	+0.470	+0.415	July.....	122	+0.433	182
April.....	+0.215	+0.165	August....	92	+0.250	212
March	−0.145	−0.075	September	62	0.000	242
February .	−0.275	−0.355	October ...	32	−0.250	272
January...	−0.295	−0.490	November.	02	−0.433	302
December.	−0.395	−0.395	December .	−28	−0.500	332

And accepting this trial oscillation, there results

$$\begin{array}{rl} & \log \\ 0.50 = & \overline{1}.6990 \\ f_4 = & \overline{5}.1995 \\ a_4 = & \overline{4}.4995 = 31\,580 \end{array} \Big\} \text{ or } IV\,S = 31\,580 \sin t.$$

While plotting and comparing it is seen that the zero of t is some 6° earlier than the mid-January period, or in this case 66° must be subtracted from all values of t, to reduce them to the Superior time scale.

Hence:

Maximum effects.				Complete oscillation, Ontario.	
Erie time.	$IV\,t'_4$	$= 151° 38'$		t	$IV\,h_5$
		$66° 00'$	log	136	+0.280
Superior time.	$IV\,t'_4$	$= 85° 38'$	$IV\,h'_4 = \overline{1}.6990 = 0.500$	106	+0.243
	$+R_5$	$50° 36'$	$+\log p_5\ \overline{1}.7482$	76	+0.140
	$IV\,t'_5$	$= 136° 14'$	$IV\,h'_5 = \overline{1}.4472 = 0.280$	46	0.000
				16	−0.140
				− 14	−0.243
				− 44	−0.280

Again, for Ontario, we have:

Local Observed Oscillation.				Computed Trial Oscillation.		
				t	$IV\,h_5$	t
May.....	+0.665	+0.665	May.....	136	+0.665	136
April.....	+0.555	+0.640	June	106	+0.576	166
March	+0.130	+0.365	July.....	76	+0.333	196
February .	−0.135	+0.075	August....	46	0.000	226
January...	−0.275	−0.300	September	16	−0.333	256
December.	−0.530	−0.575	October....	−14	−0.576	286
November.	−0.665	−0.665	November.	−44	−0.665	316

From which

$$\begin{array}{rl} & \log \\ 0.665 = & \overline{1}.8228 \\ f_5 = & \overline{5}.2711 \\ a_5 = & \overline{4}.5517 = 35\,620 \end{array} \Big\} \text{ or } V\,S = 35\,620 \sin t.$$

While plotting and comparing it is seen that the zero of its t falls 14° earlier than the mid-January period, or subtracting 74° from the t of its trial oscillation brings it also upon the Superior time scale.

In going down the chain of lakes, out of these different elements of their oscillations any desired combination can be readily made up by scaling coincident values of h and taking their algebraic sums. Of these the leading combinations have been made up and shown with the observed variation of the mean levels on Fig. 4; and, indeed, it is mainly in their graphic representation that this series of movements is best followed. However, the purpose of this study has been simply the analysis of these movements, and with the equations and the illustrations that have here been given, it is thought that there are few questions that may arise in regard to variations of supply and their effects through any chain of lakes or reservoirs that may not be readily calculated.

In contrast with these relations in the lake levels it may be interesting to note briefly the character of similar relations in the Mississippi River. There, of course, the mathematical basis is wanting. Between the lakes, it matters not what energy may be stored in the flow, it is wholly absorbed through agitation in the inert mass below. Thus the sheer fall of the Niagara River makes not the slightest difference in the relation of supply to discharge, or the rise of one lake to the other; but it would certainly be a bold assumption to start with, to hold that essentially the same thing was true for the river; and even if that were the case, it is known that the linear variation of Q to h is not correct through the great range of its flood oscillations.

While, therefore, we cannot here calculate between two points what should be the retardation R, the reduction P, or the ratio of rise p, there are at some twenty or more locations along the Lower Mississippi twenty-five to forty years of daily gauge-readings from which to determine what they are; and as all these data have been worked over, again and again, along just these lines, its positive showings at least merit a comparison with the above theoretic deductions.

They may be stated as follows: First, for the ratio of reduction P, as might be expected, it is practically unity. If there is any general decrease in the magnitude of the discharge as the flood-wave passes down, it is altogether beyond the range of observation. However, p, the ratio of rise or fall, has in general a marked difference in value between different gauges. It is a well-determined constant for each reach of the river; and while in the course of a number of years it may change to some extent with a change of regimen, in the lower Missis-

sippi it is remarkably stable. Finally, R, the retardation, is also there a constant of the reach. Its value is very different from gauge to gauge, and differs greatly per mile in different parts of the river, but once determined for a given reach it seems to be practically the same there for all floods and for all time. It is, however, measured in days and hours, where the retardation of the lakes is measured in months and years; and, unlike the retardation, which is a fraction of the cycle, the retardation in the river is the same absolute time, whether the flood-wave is long or short.

From these relations of the river it might be inferred that we are dealing with something like a limiting case of a chain of lakes, as, indeed, it is well known that we are. Were the Mississippi to run down to a zero of discharge, that would be exactly what it would show, only here their areas, some 5 to 15 miles long, and from a half to a mile wide, are almost lost sight of, in comparison with the great discharge capacity that exists between them. In general, also, it is a fact that at the highest stages the greatest velocities are found in the locations of the lakes themselves; and with their functions thus reversed, they may be much more nearly taken to represent the points of outlets. It is not, however, a part of this study to consider theories of flow: and it is enough here to note that, as the character of flow through a chain of lakes corresponds to observed facts in the river, it furnishes the basis of a radical departure from the whole series of assumptions upon which hydraulic formulas have heretofore rested.

DISCUSSION.

Mr. Wisner. GEORGE Y. WISNER, M. Am. Soc. C. E.—The members of the Society who are interested in the hydraulics of the waterways of the Great Lakes are certainly deeply indebted to the authors for their very elaborate analysis of the theoretical effect of storage on flow of water in rivers.

In order that the old adage that "figures cannot lie" may hold true in all cases, it is necessary that the premises and the data from which the formulas are deduced be strictly true, otherwise the practical application of such formulas may lead to the absurd result that actual existing conditions are impossibilities.

On page 402 it is stated that the supply to any given lake from inflow, drainage, precipitation and evaporation will vary as the sine of an arc on a circle whose whole circumference represents one year; that "the discharge from the lake is, of course, determined at any time solely by the value of h, or the level of its outlet at that time"; and that "it is sufficient to take Q as made up of a constant element Q_1, the value of discharge at an assumed level, and a variable element $c\,h$, where h gives the surface at any time above or below this arbitrary level, and c is the average change in discharge there per foot of rise or fall."

These assumptions come about as near representing the actual conditions on the lake waterways for any given year as a composite photograph of a hundred persons would represent the individual characteristics of any one of the sitters.

The formulas based on these assumptions have been derived by the use of the integral calculus, which presupposes that the supply, the discharge through the outlets, and the oscillations of the lake surfaces are regular variables; while in fact they are well known to be so irregular that it is impossible to predict their variations one month in advance. From these deduced formulas it is shown (see page 363) that the ratio of the maximum supply to the maximum discharge is independent of supply. This conclusion alone is sufficient to indicate that the results derived theoretically do not conform with the facts.

The supply is made up of the algebraic sum of the inflow from other lakes, the run-off from the water-shed, the precipitation on the lake, and the evaporation from the surface of the lake. The first three of these elements are positive and the last negative, and as a general rule it may be stated that years of maximum precipitation and inflow are years of minimum evaporation, and *vice versa*. An examination of the tables of water levels of Lakes Michigan and Huron for the past 35 years, shows that the fluctuations of the levels, due to variation in the supply and discharge, at times amount to as much as 8 ins. per month

in depth over the whole lake, corresponding for a rising stage to a positive supply of about 500 000 cu. ft. per second, and for a falling stage to a negative supply of about 200 000 cu. ft. per second, making the total variation in the supply 700 000 cu. ft. per second. The area of Lakes Michigan and Huron varies but little for different stages, and, therefore, whatever theoretical change takes place in the ratio of maximum supply to maximum discharge for different seasons or stages depends on the value (c) of the increment of change in discharge for 1 ft. change in stage—a quantity which has been assumed by the authors to be a constant, or, in other words, the ratio would be nearly constant, which the tables of water levels show is not the case. Mr Wisner.

The fundamental equation, $Q = f(h)$, in which $f(h)$ always increases and diminishes with h, is all right for reservoirs under certain conditions, but, unfortunately, these conditions do not now exist at the outlet of Lake Huron. In fact, the conditions the authors have assumed to exist are the very ones that the advocates of lake regulation are desirous of producing.

The St. Clair and Detroit Rivers, being simply connecting links between Lakes Huron and Erie, their surfaces fluctuate with the levels of the two lakes and with a slope depending upon the respective levels of each, and since the fluctuation of Lake Erie exceeds that of Lake Huron, the slope of the rivers is generally greater at the low stages of the lakes than at the high stages. A large percentage of the fall from Lake Huron to Lake St. Clair (about 5.4 ft. for mean stage) is at the outlet of Lake Huron (which at the gorge is only 750 ft. wide), and any change in this fall, due to unequal changes of levels of the respective lakes, produces very decided changes in the slope of the rapids at the outlet of Lake Huron, and a corresponding change in the volume of the discharge. The volume of discharge, therefore, not only depends on the stage at the outlet, but also on the relative elevation of the water surface of Lake St. Clair; but as the water surface of the latter follows the level of Lake Erie very closely, the discharge of Lake Huron will depend on the stage of that lake, and also on that of Lake Erie. If the level of Lake Erie be so regulated that the oscillations, aside from those due to winds, be reduced to a few inches, the area of the cross-section of the outlet of Lake Huron and the slope of the river through the gorge would depend upon the stage of water at the outlet, and the fundamental formula used by the authors would then be applicable. It needs no mathematical demonstration to establish the fact that storage is absolutely essential to any system of improvement of the lake waterways based on the regulation of lake levels. The studies which are now being made of this subject contemplate only such changes in the flow through the lake outlets as will produce the maximum benefit to commerce, by controlling the discharge at Buffalo sufficiently to maintain a nearly constant level for

Mr. Wisner. the lake, and at the same time rendering the storage in Lakes Michigan and Huron more effective, by making the discharge from Lake Huron depend on the stage of water in those lakes. The slope of the St. Clair, under present conditions, is greatest during the low-water season of the winter, but, with Lake Erie regulated, it would be a minimum in the winter, thus giving a maximum effect to the storage capacity of Lakes Michigan and Huron.

The area of the lake reservoir system above Lake Erie is such that, for an average season, the inflow through the Detroit River would raise the level of Lake Erie 22 ft. if not discharged, and the changes in the flow, which will be produced by regulation, will be so small, compared with those due to variation of precipitation and evaporation, that no injurious results can possibly arise.

It is stated that "The balance of forces which Nature has here produced in the course of long ages is one of the most marvelous features of these lakes; and a careful contemplation of it cannot fail to convince one that an almost perfect compromise has been reached between the conflicting oscillations of lake level and outlet discharge. It is difficult to see how either could be brought nearer to absolute uniformity without a resulting departure in the other which would more than offset the gain."

The same, with equal fairness, may be said of the tidal forces, which in a long series of years have produced certain depths on the sand bars which obstruct the entrances of ocean harbors; yet where would the commerce of this country be to-day if these forces, under the guiding skill of the engineer, had not been so modified that they have produced and maintained channels of more than double the natural depths? If it is true that beneficial results cannot be produced by modifying the action of the forces of Nature, the writer sees no reason why the maritime and river and harbor engineers should not retire from business.

The author of the first part of the paper advocates placing regulating works directly in the outlets of lakes, such that the piers would contract the cross-section sufficiently to raise the lake level, and, by means of needle dams, control the oscillation of the lakes.

It is certainly a fact that, if the gorge of the outlet be diminished in width, the mean level of the lake will be raised, and it is equally true that such contraction will increase the fluctuations of the lake level and thereby augment the evil which it is wished to correct, and at the same time cause great damage to the existing structures of the lake harbors.

If the regulating works should be placed at the foot of the lake, above the outlet, the water in the gorge below would rise and fall with the change in the discharge over the fixed dam, or through the regulating dam—either of which would accomplish the purpose—and thereby carry off the surplus water at the time that the lake in its

natural condition would rise, and restrict the outflow during the remainder of the season, when, under the present conditions, the discharge through the outlet is greater than the supply. Mr. Wisner.

It is stated that "The only way the effects of diversions can be counteracted is to contract the outlets, so that the flow through them at the normal mean level shall be diminished by an amount equal to the diversion."

The result of such a remedy would be worse than the evil it is intended to correct. It is a well-known fact that vessels on the lakes are constructed with reference to maximum navigable depth in the lake channels, and any improvement tending to increase the fluctuations in these depths would be very injurious to the lake commerce.

The depth, which may be obtained and maintained through our lake waterways and thence to the sea-board, is not nearly of so much importance as that it shall be made nearly constant for the season of navigation through the channels from Lake Huron to Buffalo.

This cannot be done by any contracting works in the outlets of the lakes, but it can be accomplished by regulating works at the foot of Lake Erie, leaving the outlet unobstructed to take care of the maximum discharge, which would be approximately the same as under natural conditions. With the level of Lake Erie regulated, the discharge at the outlet of Lake Huron would vary with the stage of water in the lake, and, if it should then be found that the storage capacity of Lakes Michigan and Huron was in excess of what was needed, the fluctuations of those lakes could be reduced by enlarging the cross-section of the gorge at the head of the St. Clair River.

In the improvements, by dredging, which have heretofore been made in the St. Mary's, St. Clair and Detroit rivers, the material excavated has been carefully deposited in the streams adjacent to the cuts, for the purpose of maintaining the normal cross-sections as established by Nature, and it has been shown, theoretically, by several very elaborate reports, that these improvements could to no appreciable extent lower the slopes of these waterways; yet, in spite of these theoretical impossibilities, the water level below the locks at Sault Ste. Marie is about half a foot less than for the same stage in Lake Huron before the improvements were made, and the slope from Lake Huron to Lake Erie, for an average stage of the two lakes, is about 7 ins. less than it was previous to the deepening of these waterways.

H. W. Brinckerhoff, M. Am. Soc. C. E.—It seems possible that some of these calculations of lake discharge might have to be labeled as Mr. Dunn, late weather prognosticator of the first district, used to mark his prophecies—"Subject to revision by the east wind." Of course the effect of the wind on lake discharge would be temporary and perhaps slight; but the speaker remembers hearing an eye wit- Mr. Brinckerhoff.

Mr. Brinckerhoff.

ness state that the flow of the Niagara River was, in the early part of the century, once completely checked, so that for a few hours it was possible to walk across the bed of the river above the Falls. This was caused by the wind, but not because the wind blew up the river. Lake Erie is the shallowest of all the Great Lakes, and, therefore, more easily affected by the wind, which causes at Buffalo, and presumably also at the other end of the lake, quite extensive wind tides. On this particular occasion it is probable that the water in the lake was unusually low, but, in addition to that, the wind, for some time blowing strongly from the upper end of the lake toward the lower, forced out through the Niagara River all the water that it could, and then turned around and blew the other way, and lowered the remaining water so much at the eastern end of the lake that it was below the level of the bar or shoal at the head of the Niagara River, thus stopping its flow for the time.

Mr. Johnson.

J. B. JOHNSON, M. Am. Soc. C. E.—The regulation or control of the level of the Great Lakes, while apparently a small matter—and many, perhaps, have jumped to the conclusion that it is very simple—is really a serious and difficult problem, and should be approached with extreme care, and only after a long series of physical investigations. The speaker believes that such investigations are now being undertaken by the officers who are in charge of the survey of the lakes, and that a number of years will be given to the study of this subject. Active work has already begun, tending to develop all the facts that would seem to be material. This paper lays out the ground on which will have to be based all future studies for the improvement of the lakes, their harbors and their connecting channels, and is a valuable contribution to professional literature.

On the Michigan Peninsula and around the shores of Lakes Michigan and Huron on the south, the ground is so extremely sandy that the underground flow must be very large, so that these lakes receive a great quantity of water which could never be ascertained by the ordinary method of observing run-off by gauging streams.

CORRESPONDENCE.

THOMAS P. ROBERTS, Esq.—Captain Chittenden, on page 359, says: Mr. Roberts.

"It may be accepted as a preliminary condition, to which there can be no qualification, that any alteration of natural conditions in the Great Lakes, which shall result in a material diminution of flow in the outlets during any portion of the navigation season, cannot be considered for a moment."

This bold sentence appears as a syllabus in the opening of the subject, and is well calculated to draw attention and excite discussion, for not all of his readers will agree with its sweeping conclusions.

It may be admitted that no diminution in the flow of the St. Mary's, and possibly in the Detroit River, is to be permitted during the season of navigation, *i. e.*, reducing their flow below their mean discharge, but the remark is not so fully applicable to the Niagara, which is not navigable, unless, indeed, it be shown that no contracting works to compensate for the diminished supply to Lake Ontario can be tolerated on the St. Lawrence. May it not be possible, even with the discharge of the St. Lawrence somewhat diminished, with training works to maintain its navigation unimpaired? Theoretically, at least, it does not appear to the writer to be practicable to maintain the levels of the Great Lakes at or above their mean levels without, during seasons of protracted drought, somewhat trenching upon the discharge of the outlets, yet he fancies this trenching will not actually amount to reducing their discharge to less than their extreme normal minimum flow. Their least discharge with dams should be more than their existing minimum, because the dams, by increasing the areas and capacities of the lakes, under Captain Chittenden's Fundamental Proposition I, will give to their outlets a more uniform discharge than they would have without them. However, as "we must not trust too far to keeping our pennies while buying our pies," at the same time let us not be alarmed too much concerning the outlets. If some of their water supply should happen to be shut off, it is within the range of art to compensate for such mischief.

The writer finds himself mentally noting "correct," "correct," to paragraph after paragraph of Captain Chittenden's propositions and arguments, though all the time instinctively feeling that to agree to all of them will leave the advocates of dams with not a leg to stand on, and to almost force the conclusion that nothing whatever can be done for the improvement of the Great Lakes. Captain Chittenden and Mr. Seddon deserve thanks, if for nothing else, in having demonstrated that great carefulness is to be exercised, and that only conservative ideas of lake improvement are likely to be ever realized.

Referring to one of the deductions from Proposition III, relating

Mr. Roberts. to the suggestion of storing water in Lake Michigan to make up the amount diverted for the proposed maintenance of the Chicago drainage canal, Captain Chittenden says, page 381:

"The storage of water in Lake Superior must commence by cutting off the outflow. When the stored water is run out, the total increase of flow over the normal condition will only be equal to the previous decrease. In other words, the total supply to the lakes below, upon which their mean level depends, cannot be altered a particle by storage."

Certainly the total supply to the lower lakes will not be altered a particle, but this is far from proving that the regulation of the supply from Lake Superior will not prove of benefit to Lakes Huron and Michigan. It may be found practicable to so gradually store water in Lake Superior as not to, while so doing, diminish the twin level lakes below a desirable least level. However, the amount of damage which is likely to result by reason of overflow along the shores of Lake Michigan should be known before seriously discussing the utility of that great lake as an impounding reservoir.

In the case of the improvement of a river by means of permanent dams, the engineer expects to increase its mean level, but at the same time reduce the range between its high and low stage. The only thing calling for much concern is the effect of his dams in increasing the height of the maximum rises. It cannot be said that dams at the outlets of the Great Lakes would be analogous to dams on an average river, for very likely fixed structures in the outlets of the lakes would disastrously affect the maximum levels of the lakes above them, but with a system of movable dams under proper management, the writer contends that the records of the fluctuations of the lakes during the past quarter of a century do not afford ground for alarm.

Referring to the profile of the lake oscillations, Plate XXXVI, on Lake Superior is found a conspicuous apex during the summer of 1876, so marked indeed that it becomes a matter of practical importance to know what damages would result from considering it an elevation which might hereafter be approximated, at least somewhat more frequently than in the past, if a dam were constructed on the St. Mary's River. The profile indicates, however, that for three years preceding 1876, the lowest levels of Lake Superior were above the mean level, so that the probabilities are that if an adjustable dam had been in existence during those years it would have remained down, and consequently the rise of 1876 would not have been augmented by its existence. If, in the fall of 1876, the dam had been raised, it could have held surplus water until 1879 or 1880, which (without a dam on the St. Clair River), would have assisted in maintaining Lakes Michigan and Huron. However, it is to be granted that when such large profile areas of depression as existed in the lower lakes between 1890 and 1898 are considered, it becomes manifest that something

more than storage water from Lake Superior is demanded to meet the requirements of navigation. Mr. Roberts.

For Lake Erie there appears to have been no urgent necessity for increased depth from 1875 to 1891, a period of 16 years. Let it be supposed that an adjustable dam had existed in the Niagara River during these years. It would probably have been lowered in 1882, when the indications were that the lake would not decline to mean level. It would have been partially raised in the beginning of 1889; lowered one year later early in the season, possibly without materially increasing the summer rise of 1890; again raised in the fall of 1890, and kept either altogether or partly raised, to the present season. This would have made only four maneuvers of the dam necessary in sixteen years' time, the last maneuver holding Lake Erie at, or somewhat above, its mean level for seven years, during which time it is difficult to see how the discharge of the Niagara River would have materially varied from its discharge during those years, with no dam supposed to be in existence. The summer rises on Lake Erie would have been several tenths higher with a dam than without one, but the range between high and low on the lake would, with the dam, have been greatly lessened, and a lessened range on the lake implies a more equable discharge in its outlet. Nor does the study of the profile of oscillations indicate that with adjustable dams at the outlets of any of the lakes, excepting possibly Ontario, arranged to be lowered invariably upon the advent of rises above mean lake level, would have added to their maximum rise, for the reason that in no year preceding a decided maximum on the lakes above Ontario is found, during the navigation season, a level below the mean, calling for their raising; consequently, the dams would have been down many months before the maximum floods neared the danger line. The year 1890 is an apparent exception to this rule, for Lake Erie, when a winter low water of about 1 ft. below mean level was followed six months later with a rise of 1½ ft. above mean level.

Whatever importance Captain Chittenden may attach to his own deductions from the propositions laid down by himself, it is gratifying (see page 383), if dams must be had, to find him endorsing a series of needle dams between piers extending across the Niagara, in preference to a fixed wier. The writer entertains the same preference for an adjustable structure at that point, and, in January, 1896, in a communication to Senator Brice, which was published, along with communications from others, in a report, without a document number, of a committee to the Senate, suggested pontoons between piers.

The writer never entertained the idea that the lakes could be maintained at a substantially uniform level, but believes it to be practicable to confine their range between their present mean and high-water levels. The phenomena on the lakes which will follow the

Mr. Roberts. construction of controlling works which would affect this result may indeed present novel aspects responsive to the rules laid down in the paper, but not attended with consequences, in all probability, as alarming as might be inferred by some who will grant the correctness of the propositions advanced by its authors.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E.—The general question of the effect of reservoirs as moderators of flood flows, and the use which can be made of their water registers to determine the yield of the tributary drainage areas, has been, so far as known to the writer, more thoroughly discussed in this paper than in any other place. The authors deserve great credit for the thorough manner in which they have treated the problem. Among the more interesting discussions of the general proposition with which the writer is familiar, that of Lieutenant-General Mullins, as given in his Irrigation Manual, may be mentioned. In that discussion General Mullins refers to formulas devised by General O'Connell, of the East Indian service, which, however, have not passed under the writer's notice any further than as referred to in the "Irrigation Manual." The general problem has been discussed in other places, but the present paper must be credited with being the most complete of any discussion thus far made.

Many of the propositions of the paper are fundamentally sound, although the writer cannot but think that the methods of computation used are more cumbersome than actually necessary. It is believed that an arithmetical solution could be used with considerable economy of labor and time. Under this head, it may be pointed out that the authors' use of quantities in cubic feet per second is one of the unnecessarily cumbersome features of the computation. A reduction of the data to inches on the water-shed, and a rational use of the same, would by itself simplify the treatment greatly. It is not desired, however, to specially press this point, but rather to point out what seems to be an unsound position of the authors in regard to the desirability of considering rainfall, run-off and evaporation data.

It is conceded that the uniformity of flow characterizing the discharge of the Great Lakes is due, purely, to the equalizing effect of the vast storage on the surfaces. The position of the authors apparently is, that the effect of this storage is so great that consideration of the temporary effect of rainfall and run-off from the tributary land areas is unnecessary. It seems to the writer that this is equivalent to saying that the whole is less than the sum of all its parts. Since making, two years ago, a brief study of some of the questions discussed in the paper, it has seemed to the writer an exceedingly interesting circumstance that, notwithstanding all the discussion of outflow of the Great Lakes which has taken place, no one has thus far been able to treat the rainfall and run-off part of the problem satisfactorily.

Mr. Rafter.

This is the more astonishing when is considered the number of rainfall data, applying to the basin of the Great Lakes, which are now available. A considerable number of meteorological stations have been maintained in this drainage basin by the Federal Government from about 1871, while all the States bordering on the Great Lakes have maintained State services for a number of years, thus accumulating a vast amount of meteorological data. The Lake Survey also gathered considerable useful information of this character, from about 1866 or 1867 to 1871 or 1872. To the north of the Lakes, the Dominion Government has maintained meteorological stations since about 1870. There are, therefore, fully twenty-five years' data of the rainfall of the Great Lakes Basin; and while it is true that a longer period would give more accurate means, still, inasmuch as this happens to be the kind of a problem where means are not specially important, it is believed that the data are sufficient for a rational solution. As to the proper method of treating this portion of the data, the writer would suggest the preparation of a map of the drainage basin, on which should be compiled all the rainfalls for the period covered by each record. This map should be divided into isoclimatic areas, each representing a given rainfall. With the exception of the widest portion of Lake Superior, there is no good reason for supposing that any of the lakes are so wide that the lines indicating these divisions could not extend, without material error, across the lakes, there being, thus far, absolutely no evidence derived from actual observation that the rainfall over the water areas is very materially different from that on the adjacent land areas. With such a map, and with the areas of each district carefully taken out, it would not be difficult to compute the mean rainfall for any given year over the entire drainage basin far more accurately than the run-offs through the outlets can ever be measured. Such a map may be termed an isoclimatic chart, and, with the large number of data now at hand, the sub-areas could be so apportioned that the meteorological conditions in each area would substantially correspond.

The authors have referred to the year 1895 as the one of extreme low water in all the lakes except Superior, which during that year was slowly rising. The following references to the rainfall data for the years from 1892–95, inclusive, will show not only that a fall in the lakes was inevitable during the period mentioned, but also that, on the whole, such decrease in elevation was in accordance with the clear indications of the rainfall record. As to the fact that Lake Superior was rising while the other lakes were falling, the writer at this time attempts no explanation.

Table No. 5 contains statistics of rainfall for the years 1892–1895 at a number of points either actually in or very near the Great Lakes Basin, which have been prepared for water years, each beginning with

Mr. Rafter. the month of December of the previous year, and terminating with the month of November. The water year is divided into three periods; the months from December to May, inclusive, forming what is termed a storage period; June to August, inclusive, the growing period; and September to November, inclusive, the replenishing period. Under this arrangement the water year for 1892, for instance, extends from December, 1891, to November, 1892, inclusive, and so on for the other years.

The run-offs of the growing and replenishing periods are comparatively small, and the statements of the quantities of rainfall for these periods are, therefore, omitted in the table in order to save space.

While the table contains only a few of the many records that could be cited from the Great Lakes Drainage Basin, still, inasmuch as nothing more than illustration is required, it may be considered sufficient at this time. It is easily determined that, at the stations cited, the rainfall was either very low for all the years in the period under consideration, or else that for the year 1895 it was much lower than the mean. Duluth, Minneapolis, Green Bay, Madison, Milwaukee, Chicago, Logansport, Ann Harbor, Grand Haven, Saint Ignace, Traverse City, Cleveland, Toledo, Buffalo, Winnipeg and Toronto may be cited in proof of this position.

TABLE No. 5.—RAINFALL, IN INCHES, AT VARIOUS PLACES IN OR NEAR THE GREAT LAKES BASIN.

	Storage Period. December to May (inclusive).				Total Yearly Rainfall. December to November (inclusive).				Mean of four water years.	Difference between columns 8 and 9.	Difference between columns 8 and 7.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
	1892	1893	1894	1895	1892	1893	1894	1895			
Pokegama Falls, Minn.	11.76	9.64	14.72	9.14	21.61	25.54	31.77	27.33	26.56	+ 0.77	— 4.44
Duluth, Minn.	17.96	11.08	19.44	6.44	32.13	21.58	31.75	23.46	27.23	— 3.77	— 8.29
Minneapolis, Minn.	13.78	12.80	15.66	7.72	39.43	28.27	23.85	22.69	28.56	— 5.87	— 1.16
Green Bay, Wis.	14.95	14.85	19.65	10.06	35.37	30.12	38.63	20.72	31.21	—10.49	—17.91
Madison, Wis.	18.89	13.37	10.96	5.54	37.12	31.94	24.80	11.99	26.46	—14.47	—12.81
Milwaukee, Wis.	18.17	15.69	15.94	10.34	34.88	31.89	29.54	23.12	29.86	— 6.74	— 6.42
Chicago, Ill.	16.03	13.93	14.48	9.58	36.25	26.96	27.94	27.28	29.61	— 2.33	— 0.66
Logansport, Ind.	27.26	24.45	21.11	9.08	45.14	37.89	33.51	24.04	35.15	—11.11	— 9.47
Ann Arbor, Mich.	14.50	20.54	16.63	8.92	30.38	38.05	26.60	19.02	28.50	— 9.49	— 7.58
Grand Haven, Mich.	16.57	19.02	19.84	10.89	30.47	34.49	34.99	21.94	30.47	— 8.53	—13.05
Marquette, Mich.	16.03	14.81	24.65	16.35	28.45	31.38	39.21	32.28	32.83	— 0.55	— 6.93
Saint Ignace, Mich.	12.39	15.61	17.80	11.66	29.12	31.86	34.43	25.23	30.16	— 4.93	— 9.20
Traverse City, Mich.	17.65	17.88	20.62	16.69	37.13	36.36	35.95	29.07	34.63	— 5.56	— 6.88
Cleveland, O.	19.84	19.09	15.28	9.29	37.65	32.00	28.60	24.79	30.76	— 5.97	— 3.81
Toledo, O.	17.77	10.17	14.93	9.23	37.00	21.90	22.94	22.58	26.10	— 3.52	— 0.36
Buffalo, N. Y.	22.62	20.65	22.47	14.17	47.87	36.52	40.79	29.25	38.61	— 9.36	—11.54
Winnipeg, Man.	6.65	8.25	8.55	8.18	19.31	23.41	18.19	17.22	19.53	— 2.31	— 0.97
Port Arthur, Ont.	8.84	8.48	8.20	8.76	21.50	22.37	22.07	22.67	22.15	— 0.52	+ 0.60
Toronto, Ont.	12.21	18.64	19.90	11.93	31.15	36.35	31.41	25.93	31.21	— 5.28	— 5.48

Mr. Rafter.

The data given in Table No. 5, without being exhaustive, show that the year 1895 was, generally speaking, a year of low rainfall throughout the Great Lakes Basin, and, in view of the general accord between the rainfall record and the elevation of the lakes, the writer cannot but think that the relation between the rainfall and run-off of the Great Lakes System was, in this case at any rate, considerably closer than the authors are disposed to admit.

By way of further showing the substantial accord of the water register record of surface elevation of the Great Lakes, the rainfall and the run-off, the writer will now cite some run-off records for the years 1892–95, inclusive, of streams either tributary to the Great Lakes, or immediately adjacent thereto, the first record cited being that of the Upper Mississippi reservoirs which control a drainage area of 3 265 sq. miles in Minnesota, immediately to the west of the Lake Superior drainage basin. The rainfall of the area tributary to the Upper Mississippi reservoirs, as indicated by records kept at Leach Lake, Winnibigoshish Lake and Pokegama Falls, from 1885 until the present time, is found to be, as an average, from 24 to 26 ins. per year, and very similar in its distribution and other characteristics to the rainfall of the area tributary to Lake Superior and the northern portion of Lakes Michigan and Huron; hence the run-off of the Upper Mississippi may be taken as typical of the streams tributary to Lake Superior and the northern part of Lakes Michigan and Huron. The following tabulation shows the rainfall and run-off from these reservoirs for the period under discussion:

Year.	Mean rainfall on water-shed, in inches.	Proportion of run-off to rainfall. Per cent.	Run-off, in ins., on the water-shed.
1892	21.33	20.8	4.43
1893	25.42	14.2	3.61
1894	26.63	13.6	3.62
1895	25.11	11.1	2.79
Totals	98.49	....	14.45
Means	24.62	14.7	3.61

From this tabulation is derived the interesting fact that, during the four-year period under discussion, the mean run-off of the Upper Mississippi water-shed was only 3.61 ins. on the total water-shed, although this figure is subject to correction, because the state of the reservoirs at the beginning and ending of the four-year period is not given in the report of the United States Engineers, from which these data are taken. This correction, however, cannot be very large, because the reservoirs are so operated, generally speaking, as to empty them each year. In 1895 the run-off was only 2.79 ins. The water area of the

Mr. Rafter. reservoirs themselves is 585 sq. miles, or nearly 18% of the whole. Lakes Superior, Michigan, Huron, St. Clair and Erie, have a total water surface of 87 895 sq. miles, with a total catchment area, including the surfaces of the lakes themselves, of 265 095 sq. miles. The water surface of these several lakes is, therefore, about 33% of the entire area of the basin, or nearly double the relative area of water surface and catchment area of the Upper Mississippi reservoirs. With other conditions the same, this fact might possibly lead to a somewhat greater proportion of run-off from the Great Lakes.

Another stream in the vicinity of the Great Lakes drainage area, the Des Plaines River in Illinois, may be referred to. In 1893, the mean rainfall on the basin of this river, which has a drainage area of 633 sq. miles above the point of gauging, as determined from the mean of a large number of stations in Illinois and Wisconsin, was 26.96 ins., while the run-off for the water year was 10.14 ins.; in 1894 the rainfall for the water year was 27.94 ins., and the run-off, 7.70 ins. For the water year 1895, the rainfall was 27.28 ins. The run-off record for that year is, unfortunately, incomplete, there being no record for the months of December and January, but from what is given and from the sequence of the rainfall, it is clear that the total record for the year 1895 did not exceed 2.0 ins. In 1896, the rainfall increased to 39.58 ins.; nevertheless, the run-off for that year was only 6.69 ins.

Another stream for which run-off records have been kept, the Muskingum River in Ohio, may be referred to. This stream has a water-shed above the point of gauging of 5 828 sq. miles. The headwaters are not far from Lake Erie, and are on the dividing line between the hill country of the east and the prairie country of the Mississippi Valley. It is considered that this stream, therefore, presents conditions applicable to the run-off of Ohio streams tributary to Lake Erie. The rainfall record is the mean of those kept at Akron, Canton, Newcomerstown and Worcester, and is believed to represent fairly well the mean precipitation of the Muskingum drainage area. This record was carefully worked up by Captain Chittenden himself, and presented in his admirable report on the water supply of the proposed Erie-Ohio ship canal.

In 1892, the total rainfall for the water year was 41.74 ins., and the run-off 13.38 ins. In 1893, the rainfall was 42.36 ins., and the run-off of the water year 16.20 ins. In 1894, the rainfall was 30.50 ins., and the run-off 8.70 ins., while in 1895 the rainfall was 29.84 ins., and the run-off of the water year only 4.90 ins.

The Oatka Creek, a tributary of the Genesee River, was gauged from 1890 to 1892, inclusive, the drainage area above the point of gauging being 28 sq. miles. In 1892, with a rainfall of 41.69 ins., the run-off was 15.42 ins. The Genesee River was also gauged for a portion of 1893, and for the years 1894, 1895 and 1896, the drainage area above

the point of gauging, at Mount Morris, New York, being 1 070 sq. miles. The rainfall is the mean of several stations. For the water year of 1894, the total rainfall was 47.79 ins., and the run-off 19.38 ins.; for 1895, the rainfall was 31 ins. and the run-off 6.67 ins. In 1896, the rainfall was 40.68 ins., and the run-off 12.80 ins. Hemlock Lake, another tributary of the Genesee River, with a drainage area of about 42 sq. miles, gave, with a rainfall in 1880 of 21.99 ins., a run-off of 3.4 ins. Mr. Rafter.

So far as known to the writer, the foregoing streams—either tributary or adjacent to the Great Lakes area—represent, with the exception of Skaneateles Lake, everything of which gaugings have been made. The data cited are sufficient to show that for the years 1892–95, inclusive, the run-off of the streams tributary to the Great Lakes was in accord with the water register and rainfall record. These records also show that the run-off of the land areas tributary to the Great Lakes is, in years of low rainfall, very small. Assuming that in such years the evaporation is equal to the rainfall, although probably it sometimes exceeds the latter, it is discovered at once why the land contribution to the Great Lakes has in such years so little effect. It accordingly follows that in years of low run-off from the land areas, the stage of the Great Lakes is almost entirely controlled by the inflow from above. This statement, however, does not apply to Lake Superior, which has no tributary reservoir.

In considering the water surface evaporation data, the writer must admit that the material at hand is extremely unsatisfactory. It is believed, however, that with the present state of knowledge concerning the laws of evaporation, a few years' observation at a number of points about the Great Lakes would furnish sufficient data to solve this part of the problem rationally.

Summarizing the whole matter, the writer's position is, that while the authors have handled admirably the general question of the Great Lakes storage reservoirs as moderators of the run-off, that they are still somewhat lame in assuming that their discussion of the problem is in reality complete. The more rational view is that it is desirable to correlate all the phenomena, in order to reach a complete solution.

James A. Seddon, M. Am. Soc. C. E.—It is not altogether clear just what Mr. Wisner takes exception to in the mathematical analysis, for he can hardly have so misunderstood its scope as to suppose that it was restricted simply to the annual cycle. The general method of representing supply is that of superimposed sine curves; the equations of two cycles in particular were worked out, viz., that for the n years, and the yearly cycle, and with these it was also noted that for any additional eccentricity of the seasons, similar terms would follow. There is no form of reciprocal variation that cannot be so expressed except at an arbitrary discontinuity, and, to cover the whole matter, the equation for this discontinuity was also given. In the applica- Mr. Seddon.

Mr. Seddon. tion of this, then, it would certainly have been interesting if Mr. Wisner had only undertaken to give some of those existing conditions that it proved to be impossible.

That in the application of this analysis to a chain of lakes only a single cycle was carried through in no way prevented any one from taking all others; it simply marked the way for any and all of them; while finally, in the actual example treating the mean oscillations of the Great Lakes, it was, of course, simply an interesting fact that the single annual cycle was about all that was left in these twenty-five year means. Mr. Wisner did not object to this because it did not fit the data well enough, but because it fitted the data too well.

In regard to the discharge element, it was not intended in the first place, nor is it here proposed, to enter into a general discussion of it. Those who have given any study to the great masses of such data collected will appreciate the fact that the assumptions made are sufficiently accurate for the problem; and beyond that there is no end to controversies and criticisms in it. However, Mr. Wisner's point in regard to the variation of c at the Huron outlet would be well taken if his assumptions were correct; though even in that case, if values were assigned to this variation and its effects calculated through, they would be found to be hardly worth noting.

This assumption, however, to say the least, is a peculiarly bold one. It takes only a glance at the total oscillations (Fig. 4) to show that the St. Clair level follows the Erie level very closely, but to say that the Erie level is the cause of this is quite another thing, since, for that matter, it follows the Ontario level even closer. Indeed, the St. Clair levels in this case, are wholly calculated from the probable supply of its own basin, and the effects down from Huron on it, and they have not the faintest trace of an effect up from the Erie levels in them. True, the data of St. Clair are notably deficient, and, while there has been nothing as yet published to warrant this step in Mr. Wisner's assumption, still it may, perhaps, be possible that on occasions some trace of an effect up from Erie may be detected in the St. Clair levels. To carry this up again through the St. Clair River, however, and to assume that it has a sufficient magnitude at the end to materially affect the discharge from Huron through the gorge at its head, is certainly to follow up a supposition rather further than seems wise. Indeed if Mr. Wisner will not take a caution here, he will find that Nature has an especially blunt way of her own in telling those who build on such assumptions that they are mistaken.

In regard to Mr. Rafter's suggestion of arithmetical methods, it is certainly simple enough in this way to closely approximate the effect of a variable supply on any lake, beginning with an assumed level. The only difficulty is in assuming the right level. It is true that with a given oscillation of supply, Nature might originally start from any

assumed level, but it would come in years to a balanced oscillation of the surface that had but one given value for the surface level at a given time, and long before Mr. Rafter had figured through this process he would be glad enough to drop arithmetic, and take up the methods of the calculus which determine the whole matter at once. Mr Seddon

Simply stating, however, the questions in such an analysis, say, of the twenty-five years means of the Great Lakes, involves a cumulative series of local effects transmitted down on levels that are everywhere conditioned by the ultimate balance of supply and discharge, and that can not be assumed arbitrarily. Considering the number of independent elements that go to make up these oscillations, it is certainly expecting a good deal to calculate them all, with the variations of each through the whole cycle in much less than the computations given, pages 420 to 425, where it is all worked through in the fullest detail. While, of course, it is in general a safe criticism to suggest that any method might be simplified, it is hardly a fair one unless at the same time it is also shown how.

Col. H. M. Chittenden.—In closing this discussion, the author of the first part of the paper will consider briefly the following points: Method of investigation; conclusions arrived at; character of works required. Col. Chittenden

The suggestions offered by Mr. Rafter were particularly pleasing to the author, because they present in minute detail the very programme actually followed by him until it proved impracticable. The attempt was made, with great labor and perseverance, to reach a satisfactory solution of the reservoir problem of the Great Lakes by commencing with the primary elements of S, viz., rainfall, run-off and evaporation. The records of rainfall were obtained from nearly every point in the St. Lawrence Basin where they have been officially kept during all or a portion of the twenty-five years from 1871 to 1895. Careful studies of the subject of run-off were made from the very records Mr. Rafter refers to, viz., the upper Mississippi, the Des Plaines, and the Muskingum Rivers. The subject of evaporation was considered in the light of the best available data. Repeated efforts were made, both for single years and for the means of longer periods, to harmonize these data with their effects as shown in the rise and fall of the lakes and in the flow of the outlets. The result in every case was failure. It was not possible to satisfy, within any admissible approximation, the following equation which we know must be a true one:

$$R_a + R_o - E \pm S_r = D,*$$

* While pursuing this line of investigation the author used depth in feet upon land area and water surface, as Mr. Rafter suggests. In the more general discussion, however, this was not found practicable. If was often necessary to use the same quantity with reference to different lakes and water-sheds, as where Q is taken with reference to the lakes above and below the outlet to which it refers. Moreover, the cubic foot being the unit almost universally used in American river engineering, it was thought best, for the purposes of comparison, to use it here. In the mathematical analysis the character of the unit was immaterial, as the symbols dealt with were general in character.

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R_r represents precipitation upon the surface of the lake;

R_o represents run-off from the land area, and includes, not only the discharge of the streams, but all subterranean flow into the the lakes of whatever character;

E represents evaporation from the surface of the lake;

S represents storage, and is to be considered as negative with a rising lake and positive with a falling lake, and

D represents the discharge of the outlet.

The author became thoroughly convinced that data in regard to R_r, R_o, and E in the basin of the Great Lakes are, and must ever remain, too uncertain for use in precise work. The vastness of the areas considered; the marked variations in rainfall at different points not so very far apart; the possibility that large quantities of water find their way to the lakes, as Professor Johnson suggests, by channels which can never be reached and measured; the certainty that there is an immense but indefinite body of water in the land about the lakes which flows out as the levels fall and in as they rise; and, finally, the great uncertainty in regard to evaporation;—all these drawbacks may well discourage him who tries to investigate the reservoir problem of the lakes by approaching it from this direction.

Having failed to reach any satisfactory result on these lines, the plan was adopted of studying S, not from its component parts, but from its effects, as shown in the rise and fall of the lake surfaces and in the flow of the outlets. Here, it was felt, was the nearest approach to precise data that past or future records could be expected to yield. The authors believe that this is the truly rational method of treating the problem, and that results of positive value are to be expected from it.

This is far from attempting to discredit efforts to obtain a better knowledge of R_r, R_o and E. It will always be a matter of scientific interest to reach a nearer solution of the above equation than is now possible, and assistance of great value could probably be derived from such data in any actual regulation of the lake levels.

It is hardly worth while to note in this connection Mr. Wisner's observation that the authors used integral calculus in their work and therefore had to assume a regularity of variation which is never exemplified in the actual records of lake levels. If Mr. Wisner had examined this study more closely, he would not only have avoided a misquotation of the authors, but would have seen that their method may be applied to the most irregular phenomena which the fluctuation of lake levels presents. Nowhere do the authors say, or even imply, that "the supply to any given lake from inflow, drainage, precipitation and evaporation will vary as the sine of an arc," etc. On the contrary, Mr. Seddon (page 402) says that "the actual sup-

ply in any given year can never be exactly represented by any regular variation." But by varying the length of the cycle, the most divergent cases may be studied, from the sudden rise in level due to a heavy and general rain, to those long periods of change covering several years, which the author has referred to as the cyclic fluctuation. The limitations of space made it impossible to put in a single paper a detailed explanation of the various applications of the general formula; but the method itself was indicated with sufficient clearness to be easily followed. Col. Chittenden

Mr. Wisner challenges the correctness of the statement that $\frac{Q\ (\text{maximum})}{S\ (\text{maximum})}$ is independent of S, and in so doing again misquotes the author. What is actually said (page 363) is that, "for a given cycle, R is a function of area and outlet alone, and independent of S; P also is a function of area and outlet alone," etc. This proposition is rigidly true. For a given regular cycle of change in S, whether of five days or five years, and with fixed conditions as to area and outlet, P and R will remain constant whether S be small or great.

Mr. Wisner takes exception to the author's deduction that the forces of Nature have done so much for the navigation of the Great Lakes as to make it exceedingly doubtful to what extent their action may be modified with impunity; and he holds that a general application of this principle would deprive the river and harbor engineer of his *raison d'être*. This dire foreboding is effectually dispelled by the very example which Mr. Wisner (unfortunately for his argument) has selected. The forces of Nature in tidal channels have generally been obstructive in their action; whereas, in the case cited by the author they have been exactly the reverse. Did these forces everywhere do for navigation what they have done in the Great Lakes, the work of the river and harbor engineer would indeed be greatly curtailed in scope.

The only point in Mr. Wisner's discussion which may possibly assail effectually any of the author's views is where he questions the assumption that the value of Q is "determined at any time solely by the value of h." He holds that this is not true of the outlet of Michigan-Huron—that at the lower stages of this lake the slope in the outlet is frequently greater than at the higher stages; and that, owing to this increased slope, the discharge at the lower stages is greater than at the higher. This may or may not be true. The author could not find sufficient evidence of its truth to justify him in applying a different rule to the discharge of this outlet from that of any other. He is, nevertheless, entirely open to conviction and will welcome a final determination of the matter. Those charged with this determination, however, should bear in mind that the engineering profession will be satisfied only with the most complete and convincing data. No partial or incomplete series of measurements, nor any, in fact, that fail to em-

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brace simultaneous records of discharge and stage all along the connecting link between Huron and Erie, and for every phase of the relative levels of these lakes, will be accepted as conclusive.

In regard to the effect upon slope of deepening and narrowing the channel of any stream while the area of cross-section remains the same, Mr. Wisner conveys the idea that he entertains more advanced views than other engineers. Whatever may be the "elaborate reports" to which he refers, the author recalls at least one such report, written by perhaps the ablest living authority upon the Great Lakes among Government engineers, in which the argument is enforced with great emphasis, that the effect of such a change must inevitably be to flatten the slope, and, in the case of the channels between the lakes, to draw down the levels of the lakes above. In order that no erroneous inference as to the author's position upon this point may be drawn from Mr. Wisner's remarks, the following quotation is given from page 381, where the author discusses the means of preventing the lowering of lake levels from the above causes :

"In superficial area the contraction would have to be greater than the previous enlargement, for the whole change thus made in the cross-section is in the direction of deepening and narrowing the outlet; and a deep and narrow channel will carry more water than a wide and shallow one of the same area of cross-section."

Passing now from methods to results, the author is surprised that Colonel Roberts should have found anything in the present paper to justify him in saying that, while generally agreeing with its propositions, he all the time instinctively feels that:

"To agree with all of them will leave the advocates of dams with not a leg to stand on, and to almost force the conclusion that nothing whatever can be done for the improvement of the Great Lakes."

This is certainly a very different view from that entertained by the authors themselves. If they have succeeded in correcting some fallacious ideas in regard to the practicability of eliminating the annual fluctuations in the levels of the lakes; of compensating by storage in Superior for a permanent diversion from Michigan at Chicago, and of establishing a satisfactory automatic regulation of levels in Erie by a long weir at Buffalo; they have not in any sense represented the problem of improving the navigation of the lakes by means of regulating works in the outlets as an impracticable one In order to clear up any doubt upon this point, the conclusions indicated by their investigations may here be summarized :

(1) It is neither practicable nor desirable to restrict the range of annual oscillations of the lake levels. It is the storage represented by these oscillations that furnishes the key to the whole system of lake navigation, for without it the flow in the outlets could not be maintained at the point necessary for extensive navigation.

(2) It is practicable to eliminate the cyclic fluctuations, and to

keep the levels of the lakes from ever falling below a fixed minimum plane. This would be in itself a vast gain. If lake carriers may know for a certainty that they can always have a fixed minimum depth through the connecting channels and in the harbors, and that there will not again come those periods when the mean levels will go down and down for a series of years, surely they will be relieved of one great cause of complaint. While it is probable that any practicable attainment of this end will result in an occasional increase of the annual fluctuations, there is no apparent reason why this should be objectionable, unless such increase should reach the point of causing damages. It could be utilized to swell the flow in the outlets during the navigation season and to hold it at a uniform point during that time; for again, let it be remembered, increased fluctuation of level in the lakes is a less serious matter than increased variation of discharge in the navigable outlets, particularly in that between Huron and Erie. Col. Chittenden

(3) It is entirely practicable to raise the mean levels of all the lakes, whereby an improvement of harbor and channel depths can be accomplished throughout the system. The point of difficulty in this connection will undoubtedly be so to construct the works as not to increase slopes in the channels to such a point as seriously to interfere with navigation or to necessitate the use of locks.

(4) It is utterly impracticable to prevent by means of storage in Superior a permanent lowering of mean level in the system below, which must result from a permanent diversion of supply at Chicago. The only way to prevent such a result will be to restrict the discharge in all the outlets below the points of diversion, so that at the normal mean level, it shall be equal to the previous normal mean flow less the diversion. "Regulating works" at Buffalo may, as one writer suggests, be sufficient to prevent a permanent drop in the mean level of Michigan-Huron, due to diversion at Chicago. The author does not believe it. Neither can such works have any effect upon the mean level of Ontario.

This leads to the final topic embraced in this discussion—the character of the works required for the regulation of levels of the Great Lakes. Since the completion of his paper last April, the author has had considerable correspondence and discussion upon this feature of the subject. Nothing has developed to shake his conviction that any adequate regulation of lake levels should be by means of works under the control of human agencies, as originally suggested by Colonel Roberts. A fixed work at the outlet of Erie which should give a variation of discharge of, say, 60 000 second-feet for a variation in level of say 6 ins. cannot but be disastrous, or at least very injurious, to navigation in Niagara, owing to the frequent and sudden fluctuations of level at the foot of Erie from causes in no way connected with the problem of storage in the lakes. Far from the author's plans being

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"subject to revision by the east wind," as Mr. Brinckerhoff suggests, it is the plan of the advocates of the long weir and of automatic regulation that will have to be so revised.

If the author were rewriting his paper he would modify his treatment upon pages 369 and 382 by introducing the word "fixed" before "works," and by adding a third class of works, those, namely, which are not fixed in character, but subject to direct control. He holds that there is nothing in the present state of knowledge of the physics of the Great Lakes to warrant the sweeping assertion that contracting works in the outlets may not be so constructed, as to secure all the ends above proposed without injuring the navigable condition of the channels.

The problem of improving the navigation of the Great Lakes by means of regulating works at or in the outlets is not by any means the easy and simple problem which some of the earlier expressions of the advocates of regulation would lead us to suppose. It need occasion neither surprise nor disappointment if expected results from such works are not at first fully realized. The development of such a system will very likely be tentative in character, modified by experience with works partially built, and brought to final success, not by any single stroke of engineering genius, but by the gradual contributions of many years and many minds.

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No. 840.

THE RELATION OF TENSILE STRENGTH TO COMPOSITION IN STRUCTURAL STEEL—ADDITIONAL DISCUSSION ON PAPER No. 811.*

By WILLIAM R. WEBSTER, A. C. CUNNINGHAM, H. H. CAMPBELL and WILLIAM METCALF.

PRESENTED SEPTEMBER 7TH, 1898.

WILLIAM R. WEBSTER, ESQ.—Owing to absence from the country the writer's attention was not called to this interesting paper until quite recently. The author uses values of 1 000 lbs. increase for each .01% of carbon and phosphorus, which are the averages of Mr. Campbell's values for the same elements in acid and basic open-hearth steel, as is here shown. Mr. Webster.

MR. CAMPBELL'S VALUES.

	Carbon.	Phosphorus.	Average of C. & P.
Acid steel.....	1 250	890	1 070
Basic steel.....	950	1 050	1 000
Average for acid and basic.	1 100	970	1 035

Using for round numbers 1 000 lbs.

In like manner the author's base of 40 000 lbs. is an average of the bases used by Mr. Campbell for acid and basic steel, with an addition

* "The Relation of Tensile Strength to Composition in Structural Steel," by A. C. Cunningham, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 78.

Mr. Webster.

TABLE No. 1.*—ACID OPEN-HEARTH STEEL.

Number of group.	Average Analysis. Percentage of.				Average actual ultimate, pounds per square inch.	Campbell.		Webster.			Cunningham.	
	Carbon by combustion.	Phosphorus.	Manganese.	Sulphur.		Calculated ultimate, pounds per square inch.	Difference.	Calculated ultimate, pounds per square inch.	Difference.	Difference—Constant.	Calculated ultimate, pounds per square inch.	Difference.
1.	.061	.069	.29	.071	54 700	52 120	— 2 580	53 300	—1 400	—3 400	53 000	— 1 700
6.	.087	.060	.33	.043	55 470	54 470	— 1 000	54 000	—1 470	—3 470	54 700	— 770
11.	.096	.075	.25	.060	57 200	56 890	— 310	56 430	— 770	—2 770	57 100	— 100
16.	.117	.056	.39	.028	56 950	57 740	+ 790	58 400	+1 450	— 550	57 300	+ 350
21.	.125	.047	.41	.032	56 960	57 910	+ 950	58 860	+1 900	— 100	57 200	+ 240
26.	.130	.053	.40	.042	59 170	59 050	— 120	60 540	+ 370	—1 630	58 300	— 870
31.	.143	.099	.39	.065	62 830	64 710	+ 1 880	69 940	+7 110	+5 110	64 200	+ 1 370
36.	.155	.069	.41	.034	60 940	63 500	+ 2 560	66 180	+5 240	+3 240	62 400	+ 1 460
41.	.170	.074	.43	.046	64 840	65 760	+ 920	68 670	+3 730	+1 730	64 400	— 440
46.	.207	.088	.41	.047	69 410	71 480	+ 2 070	73 500	+4 090	+2 090	69 500	+ 90
51.	.229	.065	.50	.032	70 810	72 090	+ 1 280	72 220	+1 410	— 590	69 400	— 1 410
56.	.424	.043	.57	.031	94 470	93 730	— 740	.....	.....	.....	86 700	— 7 770
61.	.113	.061	.43	.038	57 140	57 700	+ 560	59 500	+2 360	+ 360	57 400	+ 260
66.	.116	.082	.50	.069	60 870	59 930	— 940	63 290	+2 420	+ 420	59 800	— 1 070
71.	.118	.075	.42	.045	59 110	59 550	+ 440	62 070	+2 960	+ 960	59 300	+ 190
76.	.119	.065	.43	.028	61 020	58 780	— 2 240	60 320	— 700	—2 700	58 400	— 2 620
81.	.134	.045	.48	.035	58 820	58 820	0	60 810	+1 990	— 10	57 900	— 920
86.	.151	.051	.64	.033	62 650	61 410	— 1 240	65 380	+2 730	+ 730	60 200	— 2 450
91.	.183	.027	.68	.030	65 100	63 150	— 1 950	64 590	— 560	—2 560	61 000	— 4 100
96.	.212	.073	.82	.039	71 870	70 750	— 1 120	75 610	+3 740	+1 740	68 500	— 3 370
101.	.242	.076	.86	.049	78 020	74 650	— 3 370	79 360	+1 340	— 660	71 800	— 6 220
106.	.374	.057	.83	.035	90 750	88 930	— 1 820	.....	.....	.....	83 100	— 7 650
111.	.392	.029	.63	.022	98 180	88 610	— 9 570	.....	.....	.....	82 100	—16 080
116.	.333	.041	.65	.026	87 410	82 540	— 4 870	.....	.....	.....	77 400	—10 010
121.	.480	.032	.69	.022	111 740	99 530	—12 210	.....	.....	.....	91 200	—20 540
126.	.555	.109	1.13	.042	123 620	115 460	— 8 160	.....	.....	.....	106 400	—17 220

* The figures in this table are the same as those given in Table No. 1 of the original paper (*Transactions*, Am. Soc. C. E., Vol. xxxviii. p. 82), except that those under the heading WEBSTER are corrected by the writer.

for the average of .40 manganese, assumed by the author to be in this steel. The following will show how this works out:

MR. CAMPBELL'S VALUES.

	Base.	Addition for Manganese.	Base Manganese.
Acid steel.....	38 600	0	38 600
Basic steel	37 430	3 400	40 830
Average for acid and basic	38 015	1 700	39 715

Using for round numbers 40 000 lbs.

The application of these values for carbon and phosphorus, by the author, to the cases given secures very good results in most instances. The results given as being obtained from the use of the writer's tables on this series of tests, are, however, surprising, and contain many

TABLE No. 2.*—Basic Open-Hearth Steel. Mr. Webster.

Number of group.	Average Analysis. Percentage of.				Average actual ultimate. pounds per square inch.	Campbell.		Webster.		Cunningham.	
	Carbon by combustion.	Phosphorus.	Manganese.	Sulphur.		Calculated ultimate, pounds per square inch.	Difference.	Calculated ultimate, pounds per square inch.	Difference.	Calculated ultimate, pounds per square inch.	Difference.
127....	.051	.008	.25	.027	46 630	45 240	—1 390	44 620	—2 010	45 900	— 730
132....	.070	.009	.36	.050	47 360	48 090	+ 730	49 430	+2 070	47 900	+ 540
137....	.074	.021	.38	.034	49 280	49 900	+ 620	50 230	+ 950	49 500	+ 220
142....	.082	.014	.43	.038	50 770	50 350	— 420	51 280	+ 510	49 600	—1 170
147....	.105	.012	.53	.032	52 970	53 170	+ 200	54 110	+1 140	51 700	—1 270
152....	.127	.010	.43	.019	52 980	54 200	+1 220	54 050	+1 070	53 700	+ 720
157....	.144	.013	.30	.037	55 260	55 030	— 230	54 790	— 470	55 700	+ 440
162....	.165	.021	.41	.051	57 220	58 800	+1 580	60 290	+2 970	58 600	+1 380
167....	.170	.040	.42	.043	61 070	61 770	+ 700	63 280	+2 210	61 000	+ 70
172....	.183	.008	.36	.030	57 350	58 710	+1 360	57 950	— 760	59 100	+1 750
177....	.194	.023	.51	.040	61 340	62 610	+1 270	63 620	+1 010	61 700	+ 360
182....	.209	.024	.47	.053	63 220	63 800	+ 580	65 160	+1 940	63 300	+ 80
187....	.220	.015	.40	.018	63 560	63 300	— 260	62 000	—1 570	63 500	+ 60
192....	.248	.018	.40	.033	66 220	66 280	+ 60	65 440	— 780	66 600	+ 380
197....	.301	.017	.65	.029	76 890	73 340	—3 550	72 130	—4 650	71 800	+5 090
202....	.052	.019	.35	.054	49 010	47 340	—1 670	48 830	— 80	47 100	+1 910
207....	.062	.015	.21	.036	48 980	46 680	—2 300	45 630	—1 050	47 700	—1 280
212....	.076	.018	.41	.062	50 880	50 030	— 850	52 010	+1 130	49 400	—1 480
217....	.084	.021	.25	.033	50 900	49 740	—1 160	48 680	—2 220	50 500	— 400
222....	.094	.046	.43	.036	54 800	54 840	+ 40	55 310	+ 510	54 000	— 800
227....	.103	.027	.44	.054	54 950	53 790	—1 160	55 610	+ 660	53 000	—1 950
232....	.117	.053	.46	.035	57 210	58 020	+ 810	59 380	+2 170	57 000	— 210
237....	.125	.036	.54	.060	58 790	57 680	—1 110	60 450	+1 660	56 100	—2 690
242....	.131	.055	.51	.036	59 110	60 270	+1 160	62 540	+3 430	58 900	— 210
247....	.144	.028	.52	.034	58 970	58 470	— 500	60 000	+1 030	57 200	—1 270
252....	.153	.012	.46	.027	58 970	57 130	—1 840	57 460	—1 510	56 500	—2 470
257....	.173	.021	.53	.021	60 810	60 570	— 240	60 890	+ 80	59 400	—1 410
262....	.190	.047	.72	.037	66 480	66 530	+ 50	68 850	+2 370	63 700	—2 780
267....	.215	.011	.42	.024	63 470	62 580	— 890	61 580	—1 790	62 600	— 870
272....	.338	.017	.62	.026	77 950	76 600	—1 350	74 640	—3 310	75 500	—2 450

* The figures in this table are the same as those given in Table No. 1 of the original paper (*Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 83), except that those under the heading Webster are corrected by the writer.

errors. Tables Nos. 1 and 2 herewith give the results, properly estimated by the writer's tables, and corresponding differences between estimated and actual ultimate strength. It will be seen in Table No. 1, as corrected, that the writer's results are a little too high, and that, by deducting the constant of 2 000 lbs., these results are very much improved. This constant, as may be seen by referring to the writer's papers, is one that has often been suggested for use in particular mill treatment or kind of material rolled. In this case the bars were 2 ins. x $\frac{3}{8}$ in., rolled for small ingot.

The writer's method of investigation was one of successive approximations, and, as the results of each individual test were on separate cards, it was an easy matter to eliminate the effect of one element by placing all the tests having the same carbon in one pile; for in-

Mr. Webster. stance, .20 carbon. Thus the difference in the tensile strength in this particular lot of tests would not be due to carbon, at least so far as known at present, although its effect in the different proportions of other elements present might be greater in one case than in another. Assuming, however, that the difference was not due to carbon, the writer tried to find a value for the other elements that would fulfill the condition and meet the differences. After this, grouping in one pile, the cards having the same manganese, the direct influence of manganese was eliminated, and proceeding in same manner for phosphorus, and then for sulphur, the values used in the tables were arrived at. This, of course, took a great deal of time and hard work, but the values secured met the requirements of the conditions under which the work was done.

The author states:

"The later investigations of Mr. Campbell are the most complete and scientific of any that have yet been undertaken in this line. With 3 163 tests made upon 2 x $\frac{3}{8}$-in. test bars of a known and uniform condition, arranged in 272 groups of similar conditions as to strength and composition, Mr. Campbell has, by the method of least squares, arrived at the strengthening effect of the various components of steel," notwithstanding the fact that Mr. Campbell in his paper admits that in applying the method of least squares and using all the elements, his results were not intelligible, and that first one element and then another had to be eliminated from his equations, until he could obtain results that were applicable to the problem at hand. The writer does not, of course, know how familiar Mr. Cunningham and Mr. Campbell are with this method of least squares, but believes that the question is still an open one as to whether or not it is the best method to use.

Messrs. Cunningham's and Campbell's results are based on figures obtained by grouping the original tests in accordance with their chemical composition and taking the average results. The writer thinks that they will find, upon further investigation of the subject, that it is much better to take individual cases, and not an average chemical composition, as this tends to mask the influence of the elements when existing in different proportions. In the application of any of these values to individual cases, tests in which the estimated ultimate strengths do not agree at all with the tensile tests will be found from time to time, and from such apparently abnormal cases more will be learned of the influence of each element, and of proper treatment in heating and rolling, than from the tests which agree with the estimated ultimate strengths.

In order to compare the two methods in their application to individual tests, the writer has taken the 408 tests given in his previous papers, has worked out the estimated ultimate strength in each case by the author's figures, and tabulated the results in the same general form as used in the writer's previous work. The estimated ultimate strength has been deducted from the actual ultimate strength, and in

all cases in the following tables the writer's figures are, of course, the same as given in former papers. In this table the differences between the estimated and the actual ultimate strengths are subdivided as noted in the first column, and are recorded in the proper division in columns marked "Cunningham" and "Webster," corrections for thickness not being applied in either case. This table shows the necessity of considering other elements than carbon and phosphorus in estimating the strength of steel from its chemical composition. The tests recorded in the division marked "plus over 10 000 lbs." are not considered in the summary, as they are included in the division of "plus over 5 000 lbs.," and are only given to show how much out of the way are the ultimate strengths calculated by the author's formula, when applied to individual tests of steel under 75 000 lbs. tensile strength. Mr. Webster.

WEBSTER'S 408 TESTS.

Differences.	Cunningham.	Webster.
+ over 10 000 lbs.	28	0
+ " 5 000 "	175	18
+ 4 000 to 5 000 "	53	18
+ 3 000 " 4 000 "	44	26
+ 2 000 " 3 000 "	50	35
+ 1 000 " 2 000 "	34	53
Within ± 1 000 lbs.	44	106
— 1 000 to 2 000 lbs.	5	54
— 2 000 " 3 000 "	2	41
— 3 000 " 4 000 "	1	28
— 4 000 " 5 000 "	0	18
— over 5 000 "	0	11
— " 10 000 "	0	0

SUMMARY OF ABOVE.

Total + more than 1 000 lbs.	356	150
" — " " 1 000 "	8	152
Difference	+ 348	— 2
Per cent. within 1 000 lbs.	10.8	26.0
" " 2 000 "	20.3	52.2
" " 3 000 "	33.1	70.8
" " 4 000 "	44.1	84.1
" " 5 000 "	57.1	92.9

As Mr. Waddell * has compared the results in the author's Tables Nos. 1 and 2, and has given the averages of the differences under the headings, "Campbell," "Webster," "Cunningham," the writer, in view of the errors that were made by the author in working out the estimated ultimate strengths from his tables, has made those corrections in Mr. Waddell's tables and gives them on pages 454 and 455 in detail, except the averages for the plus and minus differences, which he does not understand.

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 86.

Mr. Webster.

For Acid Steel.

Character of differences.	Average Differences.		
	Campbell.	Webster.	Cunningham.
Plus differences.....	1 272 lbs.	2 856 lbs. 1 820 lbs.	566 lbs.
Minus differences.....	3 073 "	980 " 1 676 "	5 549 "

For Basic Steel.

Character of differences.	Average Differences.		
	Campbell.	Webster.	Cunningham.
Plus differences.....	742 lbs.	1 583 lbs.	1 000 lbs,
Minus differences.....	1 183 "	1 684 "	1 320 "

Average for Acid and Basic Steels.
(Mean from two preceding tables.)

Character of differences.	Average Differences.		
	Campbell.	Webster.	Cunningham.
Plus differences.....	1 007 lbs.	2 219 lbs. 1 706 lbs.	783 lbs.
Minus differences.....	2 128 "	1 332 " 1 680 "	3 435 "

Note.—The figures in the second column, under the heading "Webster," are the average differences after deducting the constant of 2 000 lbs. in acid open hearth steel, as previously referred to.

For Acid Steel (75 000 and under).

Character of differences.	Average Differences.		
	Campbell.	Webster.	Cunningham.
Plus differences.....	1 272 lbs.	2 904 lbs. 1 955 lbs.	566 lbs.
Minus differences.....	1 150 "	980 " 1 778 "	1 652 "

For Basic Steel (75 000 and under).

Character of differences.	Average Differences.		
	Campbell.	Webster.	Cunningham.
Plus differences.....	742 lbs.	1 583 lbs.	659 lbs.
Minus differences.....	1 001 "	1 223 "	1 312 "

Mr. Webster.

AVERAGE FOR ACID AND BASIC STEEL (75 000 AND UNDER).
(Mean from two preceding tables.)

Character of differences.	AVERAGE DIFFERENCES.			
	Campbell.	Webster.		Cunningham.
Plus differences.....	1 007 lbs.	2 243 lbs.	1 769 lbs.	613 lbs.
Minus differences.....	1 076 "	1 101 "	1 501 "	1 482 "

NOTE.—The figures in the second column, under the heading "Webster," are the average differences after deducting the constant of 2 000 lbs. in acid open-hearth steel as previously referred to.

Mr. Metcalf* remarks that the manufacturers can vary the strength of steel by manipulation from 50% to 100% one way or the other. The writer does not consider the criticism a fair one, as it is the manufacturers of structural steel who depend more than any one else on the relation between the chemistry and the physical properties of steel. They all apply the steel to the orders by its chemical composition, and if the tension tests of the finished product do not give the results that they expected from the analyses of the heat of steel, they at once examine into the conditions of heating and rolling; and it is by this close watch of both the chemical composition and physical treatment of the steel that they have made such great advances in its manufacture during recent years.

Mr. Metcalf fails to state that in producing the changes referred to the manufacturer would also greatly change the elastic limit, the percentage of stretch, the percentage of reduction of area, and the bending properties of the steel treated. These changes, for any given treatment, would be greater or less, in accordance with the chemical composition of the steel so treated, and would emphasize the effects of the carbon, phosphorus, manganese, etc., on the steel.

Mr. Metcalf would not consider it unreasonable to state that, if a piece of steel with a known chemical composition, and a known treatment in heating and rolling produced, say, 65 000 lbs. tensile strength, at a future time another heat of steel with exactly the same chemical composition with the same treatment in heating and rolling will produce about the same result. If there were enough standard tests to cover all cases of structural material, and they could be recorded properly, it would be merely a matter of turning to such records, in order to know what a given heat of steel should produce in the finished product, without having to interpolate for the cases not so recorded. The avoidance of the necessity for this interpolation is sought by giving values to each of the elements. The problem is certainly a very difficult one and is still somewhat obscure. Marked progress,

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 84.

Mr. Webster. however, has been made, inasmuch as it was only in 1892 that no less an authority than Mr. H. M. Howe, in summing up this whole matter said:

"If these views are correct, then, no matter how great and extended our knowledge of ultimate composition, and how vast the statistics on which our inferences are based, if we attempt to predict mechanical properties from them accurately, we become metallurgical Wigginses."

Mr. Cunningham. A. C. CUNNINGHAM, M. Am. Soc. C. E.—Mr. Webster's mathematical demonstration, attempting to show that the base and factors used in the original paper are the averages of those deduced by Mr. Campbell, proves that such is not the case.

These factors were arbitrarily assumed after numerous algebraic solutions, on the assumption that carbon and phosphorus were the only hardeners, and that the strength of a theoretical steel free from carbon and phosphorus would be 40 000 lbs. per square inch.

Several hundred trials of these factors and this base, on various kinds of steel made by a dozen different manufacturers, led to the conclusion that they were approximately right. Their application to Mr. Webster's steel was unsatisfactory, and in view of the good results obtained with many other steels, naturally leads to the conclusion that his analyses may be questioned.

In using Mr. Webster's tables two results could be generally obtained; the one giving the most nearly correct result was taken, but no further manipulation was attempted.

However, even if Mr. Webster's tables are incorrect, and also his assumptions that all elements have a strengthening effect on steel, his work has been of great value in drawing renewed attention to this subject and encouraging other investigations.

In the present case a rule has been sought that, while approximately correct, should also be easily applied. Expressed as an equation, this rule is as follows:

40 000 lbs. + 1 000 lbs. for every .01% of carbon + 1 000 lbs. for every .01% of phosphorus = ± tensile strength.

This rule is not only easily carried by the memory, but also can be applied mentally with little effort, and, as Mr. Webster admits, gives very good results in most instances. Its trial has not been confined to Mr. Campbell's tests, but, as Mr. Webster advises, it has been used extensively on individual tests.

The rule is intended to establish for the inspector and the engineer a relation between the analyses furnished by the manufacturer and the lists made; when it does not, it is time for investigation.

Except in special cases, the analysis furnished by manufacturers represents the average chemical composition of the steel, while the actual test represents the strength of a concrete portion. To attempt to establish an absolute relation between these quantities is like pacing the diameter of a circle and then computing the circumference to the thousandth part of an inch.

The rule given must stand or fall upon its merits, and these can only be determined by sufficient trial. When failures occur analysis of the test piece will show whether the rules or the test is at fault. Mr. Cunningham.

H. H. Campbell, M. Am. Soc. C. E.—The investigation which is the subject of the paper, will be found in a book by the writer, entitled "Structural Steel." On page 284 is the following sentence: "The most comprehensive and systematic study of the physical formula of steel has been carried out by W. R. Webster." A description of his method is given and then it is shown that portions of his table contain "absolutely irreconcilable conditions, for Mr. Webster takes as his starting point the dictum that carbon is a constant, and proceeds to construct a table in which it is not a constant at all, and in which it is not even constantly irregular." This point is discussed at length and is proven by quotations from Mr. Webster's results. It was the intention of the writer to treat a co-investigator with courtesy, while exercising the right to openly discuss the faults in his theories. The book did two distinct things. It declared Mr. Webster's results inconsistent with his premises, and it offered a new system of calculation by the method of least squares. Mr. Cunningham has accepted the latter method, and has undertaken to condense the results, so that engineers may have a short and simple formula by which to calculate the strength of steel when the physical composition is known. Mr. Campbell.

It is not fair to hold the writer liable for the variations in the author's results, but he does assume all responsibility for the original formula and for the method employed. He knows of no way by which the problem under consideration may be solved save by the method of least squares. There may be a better way, but, as far as the writer knows, the only other method offered has been the system of Mr. Webster, which, as shown above, gives results that contradict the original assumptions, and hence must be without scientific standing.

The method of least squares is open to the criticism of any one, but it is respectfully submitted that the remark of Mr. Webster is not criticism. He states that he does not know "how familiar Mr. Cunningham and Mr. Campbell are with the method of least squares." This has nothing to do with the case. The question is whether the work is right and the method applicable. This may be decided by a mathematical or philosophical demonstration by any one who knows more than the author or the writer.

The original formulas, as printed in the book mentioned, have been used for the last two years to calculate the tensile strength of every heat made in the Open-Hearth Department of the Pennsylvania Steel Company; and in the case of structural steels having a tensile strength of between 50 000 and 70 000 lbs. per square inch, the mathematical results are almost always within 2 000 lbs. of the figures obtained on the testing machine. So reliable have the formulas proved that a wide

Mr. Campbell. variation (say of 5 000 lbs.) is always investigated, with the result that almost always errors are discovered in the chemical determinations, or else the conditions under which the test piece has been rolled are found to have been abnormal.

The chemical errors are almost always in the carbon content. This is one of the vital errors in Mr. Webster's system, since the calculations on individual heats necessitate using color determinations, and these are not sufficiently reliable for scientific investigations.

Under the system used by the writer and endorsed by the author, each element is determined in the most approved way, and the problem is solved by a mathematical method devised for just such conditions.

Mr. Metcalf. WILLIAM METCALF, Past President, Am. Soc. C. E.—The writer agrees fully with Mr. Webster's position, that, given a certain physical specification, a steel-maker should know whether it could be filled or not, and if it could be met, he should know just about what composition of steel to use. If an engineer were to accept such steel made to a formula, without careful physical tests, he would know little about the properties of the material he received. This is all the writer claimed in discussing the paper, and the author reminded him that it was not claimed that composition and formulas would take the place of tests. This brought the author and writer into substantial agreement, and if Mr. Webster would consider the discussion as a whole he would see that he and the writer agree also.

The writer highly appreciates the great value of the work of Messrs, Webster and Campbell, and hopes they will continue it. No matter how perfectly they may work out their questions, they will never eliminate the human element unless they rework and purify the individuals as they can their heats of steel.

Mr. Webster. WILLIAM R. WEBSTER, Esq.—Mr. Metcalf's full explanation of his former remark now puts the whole matter in such shape that it is not open to any misunderstanding. The writer never advocated accepting any material without making physical tests, but has tried to find some guide for those asking for both chemical and physical tests, in order that one may agree with the other.

The point raised by the author, "that the analysis furnished by the manufacturers represents the average chemical composition of the steel," is fully appreciated. It was for this very reason that, in the writer's investigation, analyses were made of drillings taken from the broken test pieces. There were over one thousand pieces analyzed, and in many cases the drillings taken from the same test pieces were sent to the chemist under two numbers, in order to check his work.

From this it is fair to assume that the differences referred to by the author, when the ultimate strength of the writer's 408 tests were estimated by his values, are not due to errors in chemical analyses. The whole trouble is caused no doubt by the manganese being in different

proportions for the same carbons, and as the effect of manganese is not considered, just such differences are apt to result. This can be shown by any series of individual tests; the writer merely used his tests as they were at hand. Mr. Webster.

The author's comparison of calculating the circumference of a circle to the thousandth part of an inch from its paced diameter is not a parallel case. As manganese has an effect on the ultimate strength of steel, it should always be allowed for directly instead of indirectly, by giving the carbon a larger value which holds good only when there is a fixed relation between the carbon and manganese present. The steel works now use different methods of recarburization, and as this investigation is continued it will be found necessary to allow for the effect of manganese in both acid and basic steel, whether made in the open-hearth furnace or in the Bessemer converter.

In stating that the author's values for carbon and phosphorus were the averages of Mr. Campbell's values for these elements, and that his base was the average of the bases used by Mr. Campbell for acid and basic steel, the writer did not in any way mean to detract from or reflect on the value of the work, and he is glad to hear that the values were arrived at independently. As the work confirms that of Mr. Campbell it certainly adds to the value of each, the difference being only 35 lbs. in the case of each .01% of phosphorus and carbon, and 285 lbs. in the base used.

The writer cannot understand how the author gets two results in using his values. This is impossible, as will be shown later in these remarks. Mr. Campbell is to be congratulated in that he finds such a close relation between the estimated ultimate strengths and the actual strength of the 2 x $\frac{3}{8}$-in. bars rolled from the small test ingots. No doubt the tests of the finished material give correspondingly good results, although he has not said so.

"We discovered many years ago that we had been running with an error of .11% in all our low carbon determinations, and .13 in all the high steels. Thus steel of .09 carbon had been regularly determined as .20, and .50 carbon as .63. Customers ordered steel, found it right, or found it too hard or too soft, and ordered the next lot accordingly. Years had rolled by, and every customer knew just what he wanted, and could learnedly discuss the special nature of .64 and of .76 carbon. The discovery of the error in the standards was a rude shock, and the change to the new order of things was the work of many months, and a diplomatic catering to prejudice, mixed with a very strong disinclination to an open acknowledgment that we had been altogether wrong."

This quotation from Mr. Campbell's book (page 7) is a fair illustration of how little attention was paid formerly, by manufacturers or any one else, to the relation between chemical composition and the ultimate strength of steel. Whereas, to-day some mills are rolling the steel into the finished product altogether from the estimated ultimate strength based on the chemical composition of each heat. The only

Mr. Webster. tests made are on the finished material after the whole of the heat has been rolled. The steel is never allowed to cool from the time it is cast until it leaves the rolls as finished angles, bars, etc. This great advance in method of working, means a yearly saving in the coal bill of thousands of dollars, and another large saving in the cost of handling the material.

Mr. Campbell's views as to the value of the method of least squares have materially changed since he wrote his "Structural Steel," as he states on page 297:

"It is with no little disappointment that I am forced to confess that further investigation throws grave doubts on the validity of this method of least squares when applied to such a number of unknown quantities, and when any one of these quantities is of very little importance."

He now states:

"Under the system used by the writer and endorsed by Mr. Cunningham, each element is determined in the most approved way, and the problem is solved by a mathematical method devised for just such cases."

Mr. Campbell's results were unintelligible until he assumed that copper, silicon and sulphur had no effect on the ultimate strength of steel. Leaving these elements out he then obtained several results for each of the elements, in the order in which they are given below, for each class of steel. In the latter part of his investigation he considered a new series of acid and basic steels. This gave him for the acid steel 56 groups in the new series, as against 70 groups in the first series. For the basic steel he had 74 groups in the first series and 72 groups in the new series.

The values he has adopted may be summarized as follows:

Mr. Campbell's Values for Pure Iron and Increase Due to .01 Per Cent. of Carbon, Phosphorus and Manganese, in Pounds per Square Inch.

In *b* and *e* for acid steels, and in *g* and *j* for basic steels, the increase due to manganese was not considered. Mr. Campbell remarks, regarding the value of 1 444 lbs. for phosphorus in *j*,

"This value of phosphorus was not sustained by any other evidence."

"The factor *R* represents an allowance for the conditions under which the piece is rolled, whether finished hot or cold. In the present series of groups it is zero."

Acid Steels.

	C.	P.	Mn.	Pure Iron.	
(*a*) Old series	1 529 +	1 316 +	39 +	34 326	= Ultimate strength
(*b*) Old series	1 485 +	1 260		+ 33 000	= Ultimate strength
(*c*) New series	1 126 +	716 +	3 +	40 439	= Ultimate strength
(*d*) Old series	1 368 +	1 068 —	23 +	37 544	= Ultimate strength
(*e*) Both series	1 210 +	890		+ 38 600 + *R*	= Ultimate strength

BASIC STEELS.

	C.	P.	Mn.	Pure Iron.	
(*f*) Old series..	1 035 +	941 +	53 +	38 996	= Ultimate strength
(*g*) Old series..	1 085 +	1 200	+	40 000	= Ultimate strength
(*h*) New series..	935 +	939 +	114 +	36 335	= Ultimate strength
(*i*) Old series..	1 035 +	941 +	53 +	38 996	= Ultimate strength
(*j*) Both series.	998 +	1 444	+	39 987	= Ultimate strength
(*k*) Both series..	950 +	1 050 +	85 +	37 430 + *R*	= Ultimate strength

Mr. Campbell estimated the ultimate strength of each of his groups by several of the above values, and the difference between the estimated ultimate strengths and the average actual ultimate strengths of the groups were small in most cases for each class of steel, but the best results were obtained by the last values given in the above tables.

The value of any one of the elements, carbon, phosphorus or manganese, and the value of pure iron, seems to depend on the number of tests considered. If the effect of each element is constant and not dependent on the amount of other elements present, this should not be the case. There seems to be a disturbing element, running all through the successive values arrived at, which strongly indicates that the values of carbon, phosphorus and manganese are not constant under all conditions of the series of tests in either the acid or the basic steels considered.

The writer does not consider that the results obtained justify Mr. Campbell in taking the strong stand that manganese does not affect the ultimate strength of acid open-hearth steel until it is raised to over .60 per cent. In this class of steel he uses 1 210 lbs. for each .01% of carbon, instead of 950 lbs. as in the basic steel. This increase of 260 lbs. for each .01% of carbon represents an increase of 27% in its value, and it includes the effect of the manganese and part of the effect of the phosphorus. This is indicated by the lower value of 890 lbs. instead of 1 050 lbs. for phosphorus, being a reduction of 160 lbs. for each .01%, or a reduction of 15%, and the manganese is not considered at all, being a reduction of 85 lbs. for each .01% of that element.

The accuracy of Mr. Campbell's work in using the method of least squares in arriving at these values is not disputed, but his results indicate strongly that the amounts of the elements present have an indirect effect on each other, and that this is shown by the different values arrived at for each element. If Mr. Campbell's position is correct, in that the method of least squares would not give intelligible results, when he considered the effect of copper, silicon and sulphur with the other elements, on account of the copper, silicon and sulphur having very little effect on the ultimate strength of the steel, it certainly casts a reflection on the applicability of this method to the problem in hand.

Mr. Webster. as it presupposes a knowledge of the effects of three of the unknown quantities under consideration.

In his investigation the writer found that the indications were that the effect of carbon was 800 lbs. per .01% and that of phosphorus 800 lbs. per .01%, when in the presence of .06, .07 and .08% carbon, but, as the carbon increased, the effect of phosphorus increased up to .15% carbon, where .01% phosphorus equaled 1 500 lbs. per square inch. This is shown in Table No. 3.

TABLE No. 3.—Webster's Values for Phosphorus in the Presence of .05 to .15 Per Cent. Carbon, inclusive.

Per Cent.	Carbon .05, .06, .07, .08.	Carbon .09	Carbon .10	Carbon .11	Carbon .12	Carbon .13	Carbon .14	Carbon .15 and over.
P .00...	0	0	0	0	0	0	0	0
" .005 .	400	450	500	550	600	650	700	750
" .01...	800	900	1 000	1 100	1 200	1 300	1 400	1 500
" .015..	1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250
" .02...	1 600	1 800	2 000	2 200	2 400	2 600	2 800	3 000
" .025..	2 000	2 250	2 500	2 750	3 000	3 250	3 500	3 750
" .03...	2 400	2 700	3 000	3 300	3 600	3 900	4 200	4 500
" .035..	2 800	3 150	3 500	3 850	4 200	4 550	4 900	5 250
" .04...	3 200	3 600	4 000	4 400	4 800	5 200	5 600	6 000
" .045..	3 600	4 050	4 500	4 950	5 400	5 850	6 300	6 750
" .05...	4 000	4 500	5 000	5 500	6 000	6 500	7 000	7 500
" .055..	4 400	4 950	5 500	6 050	6 600	7 150	7 700	8 250
" .06...	4 800	5 400	6 000	6 600	7 200	7 800	8 400	9 000
" .065..	5 200	5 850	6 500	7 150	7 800	8 450	9 100	9 750
" .07...	5 600	6 300	7 000	7 700	8 400	9 100	9 800	10 500
" .075..	6 000	6 750	7 500	8 250	9 000	9 750	10 500	11 250
" .08...	6 400	7 200	8 000	8 800	9 600	10 400	11 200	12 000
" .085..	6 800	7 650	8 500	9 350	10 200	11 050	11 900	12 750
" .09...	7 200	8 100	9 000	9 900	10 800	11 700	12 600	13 500
" .095..	7 600	8 550	9 500	10 450	11 400	12 350	13 300	14 250
" .10...	8 000	9 000	10 000	11 000	12 000	13 000	14 000	15 000
.001 P =	80 lbs.	90 lbs.	100 lbs.	110 lbs.	120 lbs.	130 lbs.	140 lbs.	150 lbs.

In regard to manganese the effect did not seem to be constant per unit.*

"The effect per unit of manganese seems to decrease as the percentage of this element increases. For instance, steels of 0.20 and 0.30 Mn. show greater difference in ultimate strength than steels of 0.50 to 0.60 Mn., all other elements being the same. I have endeavored to cover this point irrespective of the percentage of carbon or phos-

* "Observations on the Relations between the Chemical Constitution and Physical Character of Steel," by William R. Webster, *Transactions, American Institute of Mining Engineers*, Vol. xxi, 1892-3, p. 767.

phorus, but upon further investigation it may be necessary to take one or both of these elements into account in estimating the effect of high and low manganese. Mr. Webster.

"Assuming the first addition of 0.15% manganese to increase the ultimate strength 3 600 lbs, we have:

"Increase in Ultimate Strength by Successive Increments of Manganese.

"Manganese, per cent.		Increase in ultimate strength.	Total increase in ultimate strength from 0 Manganese.
From	To	Lbs. per square inch.	Lbs. per square inch.
0.00	0.15	3 600	3 600
0.15	0.20	1 200	4 800
0.20	0.25	1 100	5 900
0.25	0.30	1 000	6 900
0.30	0.35	900	7 800
0.35	0.40	800	8 600
0.40	0.45	700	9 300
0.45	0.50	600	9 900
0.50	0.55	500	10 400
0.55	0.60	500	10 900
0.60	0.65	500	11 400"

When sulphur was not considered as increasing the ultimate strength of steel, the writer found that the indications were that the ultimate strength of pure iron without carbon, phosphorus or silicon would be about 38 000 lbs. per square inch. To this he added the value of carbon and phosphorus, and gave the results in tabulated form for convenience in using, to estimate ultimate strength (see Table No. 4).

On the top line of Table No. 4, opposite .000 phosphorus, is given the strength due to pure iron, and an addition of 800 lbs. for each .01% carbon ; as, for instance, under .10 carbon, we have 38 000 plus 10 × 800 equals 46 000. Then on the other horizontal lines, opposite .01 to .10% phosphorus, the additions have been made for the values of phosphorus given in Table No. 3. For instance, .06% of phosphorus with .06, .07 or .08% carbon 4 800 lbs. are added, but with .06% of phosphorus with 11% carbon 6 600 lbs. are added, and with .15% carbon 9 000 lbs. are added for the same amount of phosphorus.

"In basic open-hearth steel, we have deducted 2 100 lbs. from the estimated ultimate strength; this has given fair results, but the amount of deduction may have to be modified in using the new table.

"From the results obtained, I believe that I am safe in saying that in all rolled steel the quality depends on the size of the bloom, or ingot, from which it is rolled, the work put on it, the temperature at which it is finished, and the chemical composition of the steel ; that is, a table of this kind could be used for beams, angles, bars, etc. For instance, a 6 x 6 x $\frac{3}{4}$-in. angle, with a given chemical composition, might give 4 000 pounds higher ultimate strength than indicated by my table ; but by making this allowance, the table could be used to advantage to show what ultimate strength another heat of steel with different chemical composition would give if rolled into the same sized angle. I trust that this point is clear, and that some of the shape mills will take the matter up and let us hear from them." *

* W. R. W., *Transactions*, A. I. M. E., 1893, Vol. xxiii, p. 115.

TABLE No. 4.—Estimated Ultimate Strengths by Webster's Values.

Base = 38 000 lbs. per square inch for iron.
Increase of 800 lbs. for each .01 carbon.
" 800 lbs. to 1 500 lbs. for each .01 phosphorus, as explained.

Carbon.	.06	.07	.08	.09	.10	.11	.12	.13	.14	.15	.16	.17	.18	.19	.20	.21	.22	.23	.24	.25
Phos. .000...	42 800	43 600	44 400	45 200	46 000	46 800	47 600	48 400	49 200	50 000	50 800	51 600	52 400	53 200	54 000	54 800	55 600	56 400	57 200	58 000
" .005...	43 200	44 000	44 800	45 650	46 500	47 350	48 200	49 050	49 900	50 750	51 550	52 350	53 150	53 950	54 750	55 550	56 350	57 150	57 950	58 750
" .01....	43 600	44 400	45 200	46 100	47 000	47 900	48 800	49 700	50 600	51 500	52 300	53 100	53 900	54 700	55 500	56 300	57 100	57 900	58 700	59 500
" .015...	44 000	44 800	45 600	46 550	47 500	48 450	49 400	50 350	51 300	52 250	53 050	53 800	54 650	55 450	56 250	57 050	57 850	58 650	59 450	60 250
" .02....	44 400	45 200	46 000	47 000	48 000	49 000	50 000	51 000	52 000	53 000	53 800	54 600	55 400	56 200	57 000	57 800	58 600	59 400	60 200	61 000
" .025...	44 800	45 600	46 400	47 450	48 500	49 550	50 600	51 650	52 700	53 750	54 550	55 350	56 150	56 950	57 750	58 550	59 350	60 150	60 950	61 750
" .03....	45 200	46 000	46 800	47 900	49 000	50 100	51 200	52 300	53 400	54 500	55 300	56 100	56 900	57 700	58 500	59 300	60 100	60 900	61 700	62 500
" .035...	45 600	46 400	47 200	48 350	49 500	50 650	51 800	52 980	54 100	55 250	56 050	56 850	57 650	58 450	59 250	60 050	60 850	61 650	62 450	63 250
" .04....	46 000	46 800	47 600	48 800	50 000	51 200	52 400	53 600	54 800	56 000	56 800	57 600	58 400	59 200	60 000	60 800	61 600	62 400	63 200	64 000
" .045...	46 400	47 200	48 000	49 250	50 500	51 750	53 000	54 250	55 500	56 750	57 550	58 350	59 150	59 950	60 750	61 550	62 350	63 150	63 950	64 750
" .05....	46 800	47 600	48 400	49 700	51 000	52 300	53 600	54 900	56 200	57 500	58 300	59 100	59 900	60 700	61 500	62 300	63 100	63 900	64 700	65 500
" .055...	47 200	48 000	48 800	50 150	51 500	52 850	54 200	55 550	56 900	58 250	59 050	59 850	60 650	61 450	62 250	63 050	63 850	64 650	65 450	66 250
" .06....	47 600	48 400	49 200	50 600	52 000	53 400	54 800	56 200	57 600	59 000	59 800	60 600	61 400	62 200	63 000	63 800	64 600	65 400	66 200	67 000
" .065...	48 000	48 800	49 600	51 050	52 500	53 950	55 400	56 850	58 300	59 750	60 550	61 350	62 150	62 950	63 750	64 550	65 350	66 150	66 950	67 750
" .07....	48 400	49 200	50 000	51 500	53 000	54 500	56 000	57 500	59 000	60 500	61 300	62 100	62 900	63 700	64 500	65 300	66 100	66 900	67 700	68 500
" .075...	48 800	49 600	50 400	51 950	53 500	55 050	56 600	58 150	59 700	61 250	62 050	62 850	63 650	64 450	65 250	66 050	66 850	67 650	68 450	69 250
" .08....	49 200	50 000	50 800	52 400	54 000	55 600	57 200	58 800	60 400	62 000	62 800	63 600	64 400	65 200	66 000	66 800	67 600	68 400	69 200	70 000
" .085...	49 600	50 400	51 200	52 850	54 500	56 150	57 800	59 450	61 100	62 750	63 550	64 350	65 150	65 950	66 750	67 550	68 350	69 150	69 950	70 750
" .09....	50 000	50 800	51 600	53 300	55 000	56 700	58 400	60 100	61 800	63 500	64 300	65 100	65 900	66 700	67 500	68 300	69 100	69 900	70 700	71 500
" .095...	50 400	51 200	52 000	53 750	55 500	57 250	59 000	60 750	62 500	64 250	65 050	65 850	66 650	67 450	68 250	69 050	69 850	70 650	71 450	72 250
" .10....	50 800	51 600	52 400	54 200	56 000	57 800	59 600	61 400	63 200	65 000	65 800	66 600	67 400	68 200	69 000	69 800	70 600	71 400	72 200	73 000
.001 Phos. =	80 lbs	80 lbs	80 lbs	90 lbs	100 lbs	110 lbs	120 lbs	130 lbs	140 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs	150 lbs

"When rolling heavy steel plates, trouble is often caused by finishing them at too high a temperature, which gives a material with crystalline fracture, poor reduction of area, and poor bends. In order to guard against this and control the finishing temperature, we use very light draughts in rolling, and produce as good results in heavy plates as in light ones. Too much importance cannot be given to the heat-treatment of steel. Mr. H. M. Howe's recent experiments on this subject are of the greatest value, and it is to be hoped that they will be continued on a larger scale in connection with the work of rolling and forging."* Mr. Webster.

Mr. Campbell's criticism relates more to the arrangement in tabulated form of the values arrived at by the writer for each element than to the values themselves. He, of course, had the option of using these values in either form, and one or two trials would have shown that each gives the same results, and that the tables save time and work.

In order to answer Mr. Campbell in detail, it is necessary to give his criticism in full. He states in his "Structural Steel," on pages 284, 285 and 286 :

"The most comprehensive and systematic study of the physical formula of steel has been carried out by W. R. Webster. He has used the long and laborious method of successive approximations, and, by cutting and trying, has found the effect of each element upon the ultimate strength, as well as the effect of the thickness and finishing temperature. The results are given by him as follows."

He then refers to the writer's values as already given, and continues as follows:

"An examination of these figures reveals two absolutely irreconcilable conditions, for Mr. Webster takes as his starting point the dictum that carbon is a constant, and proceeds to construct a table in which it is not a constant at all, and in which it is not even constantly irregular. By his own calculation a steel of .06% phosphorus and .10% of carbon is strengthened 1 400 lbs. by the addition of .01 % of carbon, while with .10% phosphorus it is strengthened 1 800 lbs. by the same addition. Assuredly, this is not a constant effect. Moreover, carbon does not even have a constant effect with the same content of other metalloids, for, with .10% of phosphorus, an increase in carbon from .07 to .08% raises the strength 800 lbs., while an increase from .08 to .09% strengthens it 1 800 lbs.

"It would be just as correct to conclude from these results that phosphorus is a constant and carbon a variable, as to say that carbon is a constant and phosphorus a variable. The changing values which it would be necessary to assign to carbon to fulfill the first assumption would be no more arbitrary and hypothetical than the changing values assigned to phosphorus by Mr. Webster, or the changing values which he has assigned to manganese. Thus the table which has been given is entirely indecisive, since it can be translated into two diametrically opposite readings, and it must be acknowledged that one empirical formula is as good as another, provided the same answers are obtained from both.

"This curious contradiction of the premises by the conclusion can only arise from some erroneous hypothesis in the values assigned to the different elements, for in the construction of such equations it is plain that an error in one factor must be atoned for by an opposite and

* W. R. W., *Journal of the Iron and Steel Institute*, 1894, No. 1, p. 335.

Mr. Webster. equal error in another factor. If this reasoning be true, then very little faith can be attached to the formula as an expression of fundamental laws, however accurately the mathematical results may coincide with observations.

"It is to be regretted that the earnest endeavor of Mr. Webster to write the physical formula should have been hampered by the necessity of working on sheared plates, which are finished under greater variations of temperature than angles or bars, and furthermore that these plates were of basic Bessemer steel, a material which would not be chosen for its regularity. By correcting for thickness and finishing temperature, Mr. Webster has shown that about 90% of the heats investigated came within 5 000 lbs. per square inch of what his equation calls for.

"This is a very satisfactory result, and it is not in a spirit of hypercriticism (for my own results, to be given later, display examples of the same character), but from a strictly scientific point of view, that attention is called to the very unpleasant corollary that one charge out of every ten does not give results within 5 000 lbs. Some of these undoubtedly are vitiated by wrong chemical determinations, for the carbon was determined by color, and this gives only approximate results; on others there might well be an error in estimating the finishing temperature; on others there would be mistakes in measuring and testing; while some pieces, perhaps, did actually show those peculiarities which we call abnormal, which are ascribed sometimes to oxide of iron, sometimes to nitrogen, and not infrequently to the devil, but which grow less numerous as we learn more of our art.

"I cannot believe that the complicated formula of Mr. Webster represents actual conditions, and the remainder of this chapter will attempt to show that a reasonably accurate empirical equation of steel may be written without the introduction of such manifold variations, and by the use of constant values for each element within the limits usually obtaining in structural metal. It will also be shown that the first increments of manganese do not add greatly to the strength of steel, since low-manganese metal is stronger than would be indicated by a formula that applies to steels containing higher percentages of this element."

Mr. Campbell ignores the facts, that much of the writer's work was on universal mill plate, that many heats of open-hearth steel were included in the investigation, and that the chemist's work was checked from time to time by sending him duplicate sets of drillings under different numbers. The color carbon determinations were used as is the general practice at Steelton and other works.

It is hardly necessary to work out the cases referred to by Mr. Campbell after the full explanations already given as to how the writer's tables were constituted, but they are as follows:

Percentage of C.	Percentage of P.	Pure Iron.	Additions for carbon.	Additions for phosphorus.		Difference.
.10	.06	38 000 +	800 × 10 +	1 000 × 06	= 52 000	.....
.11	.06	38 000 +	800 × 11 +	1 100 × 06	= 53 400	1 400
.10	.10	38 000 +	800 × 10 +	1 000 × 10	= 56 000	.....
.11	.10	38 000 +	800 × 11 +	1 100 × 10	= 57 800	1 800
.07	.10	38 000 +	800 × 07 +	800 × 10	= 51 600	.....
.08	.10	38 000 +	800 × 08 +	800 × 10	= 52 400	800
.08	.10	38 000 +	800 × 08 +	800 × 10	= 52 400	.....
.09	.10	38 000 +	800 × 09 +	900 × 10	= 54 200	1 800

In each of these cases the difference in ultimate strength due to .01% carbon is 800 lbs., but to this is to be added the difference due to phosphorus in the presence of the higher carbons, which, in the first case amounts to 600 lbs., in the second to 1 000, while in the third there is no difference, as the carbons are .08% and below. In the last case the difference is 1 000 lbs. This gives a total difference due to carbon and phosphorus of 1 400, 1 800, 800 and 1 800, respectively. Now, reference to Table No. 4 will show the same strength due to pure iron, carbon and phosphorus, with the same difference as shown above. Mr. Webster.

There is nothing in all this that is "absolutely irreconcilable," nor is there a case where the "results contradict the premises." The values used are those which gave the best results after many trials on over 1 000 test pieces used in this investigation, besides hundreds of trials on the steel graded in the course of the routine every-day work at the mill.

As Mr. Campbell does not believe that the writer's tables represent actual conditions, the writer would ask him if any one of the elements had a different effect per unit, due to a larger or smaller amount of the other elements present, what value would be arrived at by the method of least squares? Would it not be the average value of this element? If this is the case, why is he so positive that the writer's values are not right, and that the effect of phosphorus is not greater as the carbon increases? Also, that the effects of the other elements are always the same per unit, no matter in what amount they may be present in the steels under consideration.

In other portions of his book Mr. Campbell is not so positive in his statements on this point and others referred to above, as will be seen by the following:

"It would seem, therefore, that the regularly increasing banefulness of phosphorus as the carbon is raised does not portray any change in nature, but that although the effect of the metalloid in lower steels is obscured, its character is the same. No line can be drawn that can be called the limit of safety, since no practical test has ever been devised which completely represents the effect of incessant tremor. For common structural material the critical content has been placed at 10% by general consent, but this is altogether too high for railroad bridge work. All that can be said is that safety increases as phosphorus decreases, and the engineer may calculate just how much he is willing to pay for greater protection from accident. * * *

"It is certain that carbon increases the strength of steel when present in small proportions, but that after a certain content is reached (say about 1.00%) there is no increase in cohesive power from a further addition. It will also be granted that this point is not a sudden break in the line, but that the effect of each unit of carbon decreases as it is approached. If this relation holds good throughout the whole series of alloys, then each successive increment of carbon will have a less effect from the starting point of pure iron.

"It is also possible, for the same reasons, that every other metalloid will follow the same rule, so that the influence of each separate alloyed

Mr Webster.

TABLE No. 7.—Comparison of Estimated Ultimate Strengths, In this Table Manga-

		Carbon =	.06	.07	.08	.09	.10	.11	.12	.13	.14
Phos.	.000...	Webster	49 300	50 100	50 900	51 700	52 500	53 300	54 100	54 900	55 700
"	" ...	Campbell....basic.	46 530	47 480	48 420	49 380	50 330	51 280	52 230	53 180	54 130
"	" ...	Campbell.... acid.	45 860	47 070	48 280	49 490	50 700	51 910	53 120	54 330	55 540
"	" ...	Cunningham	46 000	47 000	48 000	49 000	50 000	51 000	52 000	53 000	54 000
"	.01...	Webster	50 100	50 900	51 700	52 600	53 500	54 400	55 300	56 200	57 100
"	" ...	Campbell....basic.	47 580	48 530	49 480	50 430	51 380	52 330	53 280	54 230	55 180
"	" ...	Campbell.... acid.	46 750	47 960	49 170	50 380	51 590	52 800	54 010	55 220	56 430
"	" ...	Cunningham	47 000	48 000	49 000	50 000	51 000	52 000	53 000	54 000	55 000
"	.02...	Webster	50 900	51 700	52 500	53 500	54 500	55 500	56 500	57 500	58 500
"	" ...	Campbell....basic.	48 630	49 580	50 530	51 480	52 430	53 380	54 330	55 280	56 230
"	" ...	Campbell.... acid.	47 640	48 850	50 060	51 270	52 480	53 690	54 900	56 110	57 320
"	" ...	Cunningham	48 000	49 000	50 000	51 000	52 000	53 000	54 000	55 000	56 000
"	.03...	Webster	51 700	52 500	53 300	54 400	55 500	56 600	57 700	58 800	59 900
"	" ...	Campbell....basic.	49 680	50 630	51 580	52 530	53 480	54 430	55 380	56 330	57 280
"	" ...	Campbell.... acid.	48 530	49 740	50 950	52 160	53 370	54 580	55 790	57 000	58 210
"	" ...	Cunningham	49 000	50 000	51 000	52 000	53 000	54 000	55 000	56 000	57 000
"	.04...	Webster	52 500	53 300	54 100	55 300	56 500	57 700	58 900	60 100	61 300
"	" ...	Campbell....basic.	50 730	51 680	52 630	53 580	54 530	55 480	56 430	57 380	58 330
"	" ...	Campbell.... acid.	49 420	50 630	51 840	53 050	54 260	55 470	56 680	57 890	59 100
"	" ...	Cunningham	50 000	51 000	52 000	53 000	54 000	55 000	56 000	57 000	58 000
"	.05...	Webster	53 300	54 100	54 900	56 200	57 500	58 800	60 100	61 400	62 700
"	" ...	Campbell....basic.	51 780	52 730	53 680	54 630	55 580	56 530	57 480	58 430	59 380
"	" ...	Campbell.... acid.	50 310	51 520	52 730	53 940	55 150	56 360	57 570	58 780	59 990
"	" ...	Cunningham	51 000	52 000	53 000	54 000	55 000	56 000	57 000	58 000	59 000
"	.06...	Webster	54 100	54 900	55 700	57 100	58 500	59 900	61 300	62 700	64 100
"	" ...	Campbell....basic.	52 830	53 780	54 730	55 680	56 630	57 580	58 530	59 480	60 430
"	" ...	Campbell.... acid.	51 200	52 410	53 620	54 830	56 040	57 250	58 460	59 670	60 880
"	" ...	Cunningham	52 000	53 000	54 000	55 000	56 000	57 000	58 000	59 000	60 000
"	.07...	Webster	54 900	55 700	56 500	58 000	59 500	61 000	62 500	64 000	65 500
"	" ...	Campbell....basic.	53 880	54 830	55 780	56 730	57 680	58 630	59 580	60 530	61 480
"	" ...	Campbell.... acid.	52 090	53 300	54 510	55 720	56 930	58 140	59 350	60 560	61 770
"	" ...	Cunningham	53 000	54 000	55 000	56 000	57 000	58 000	59 000	60 000	61 000
"	.08...	Webster	55 700	56 500	57 300	58 900	60 500	62 100	63 700	65 300	66 900
"	" ...	Campbell....basic.	54 930	55 880	56 830	57 780	58 730	59 680	60 630	61 580	62 530
"	" ...	Campbell.... acid.	52 980	54 190	55 400	56 610	57 820	59 030	60 240	61 450	62 660
"	" ...	Cunningham	54 000	55 000	56 000	57 000	58 000	59 000	60 000	61 000	62 000
"	.09...	Webster	56 500	57 300	58 100	59 800	61 500	63 200	64 900	66 600	68 300
"	" ...	Campbell....basic.	55 980	56 930	57 880	58 830	59 780	60 730	61 680	62 630	63 580
"	" ...	Campbell.... acid.	53 870	55 080	56 290	57 500	58 710	59 920	61 130	62 340	63 550
"	" ...	Cunningham	55 000	56 000	57 000	58 000	59 000	60 000	61 000	62 000	63 000
"	.10...	Webster	57 300	58 100	58 900	60 700	62 500	64 300	66 100	67 900	69 700
"	" ...	Campbell....basic.	57 030	57 980	58 930	59 880	60 830	61 780	62 730	63 680	64 630
"	" ...	Campbell.... acid.	54 760	55 970	57 180	58 390	59 600	60 810	62 020	63 230	64 440
"	" ...	Cunningham	56 000	57 000	58 000	59 000	60 000	61 000	62 000	63 000	64 000
.001	Phos. =	Webster	80	80	80	90	100	110	120	130	140
"	" =	Campbell....basic.	105	105	105	105	105	105	105	105	105
"	" =	Campbell.... acid.	89	89	89	89	89	89	89	89	89
"	" =	Cunningham	100	100	100	100	100	100	100	100	100

BY METHODS OF WEBSTER, CAMPBELL AND CUNNINGHAM. Mr. Webster.

nese is .40 in all cases.

.15	.16	.17	.18	.19	.20	.21	.22	.23	.24	.25	.26	.27	.28
56 500	57 300	58 100	58 900	59 700	60 500	61 300	62 100	62 900	63 700	64 500	65 300	66 100	66 900
55 080	56 030	56 980	57 930	58 880	59 830	60 780	61 730	62 680	63 630	64 580	65 530	66 480	67 430
56 750	57 960	59 170	60 380	61 590	62 800	64 010	65 220	66 430	67 640	68 850	70 060	71 270	72 480
55 000	56 000	57 000	58 000	59 000	60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000
58 000	58 800	59 600	60 400	61 200	62 000	62 800	63 600	64 400	65 200	66 000	66 800	67 600	68 400
56 130	57 080	58 030	58 980	59 930	60 880	61 830	62 780	63 730	64 680	65 630	66 580	67 530	68 480
57 640	58 850	60 060	61 270	62 480	63 690	64 900	66 110	67 320	68 530	69 740	70 950	72 160	73 370
56 000	57 000	58 000	59 000	60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000
59 500	60 300	61 100	61 900	62 700	63 500	64 300	65 100	65 900	66 700	67 500	68 300	69 100	69 900
57 180	58 130	59 080	60 030	60 980	61 930	62 880	63 830	64 780	65 730	66 680	67 630	68 580	69 530
58 530	59 740	60 950	62 160	63 370	64 580	65 790	67 000	68 210	69 420	70 630	71 840	73 050	74 260
57 000	58 000	59 000	60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000
61 000	61 800	62 600	63 400	64 200	65 000	65 800	66 600	67 400	68 200	69 000	69 800	70 600	71 400
58 230	59 180	60 130	61 080	62 030	62 980	63 930	64 880	65 830	66 780	67 730	68 680	69 630	70 580
59 420	60 630	61 840	63 050	64 260	65 470	66 680	67 890	69 100	70 310	71 520	72 730	73 940	75 150
58 000	59 000	60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000
62 500	63 300	64 100	64 900	65 700	66 500	67 300	68 100	68 900	69 700	70 500	71 300	72 100	72 900
59 280	60 230	61 180	62 130	63 080	64 030	64 980	65 930	66 880	67 830	68 780	69 730	70 680	71 630
60 310	61 520	62 730	63 940	65 150	66 360	67 570	68 780	69 990	71 200	72 410	73 620	74 830	76 040
59 000	60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000
64 000	64 800	65 600	66 400	67 200	68 000	68 800	69 600	70 400	71 200	72 000	72 800	73 600	74 440
60 330	61 280	62 230	63 180	64 130	65 080	66 030	66 980	67 930	68 880	69 830	70 780	71 730	72 680
61 200	62 410	63 620	64 830	66 040	67 250	68 460	69 670	70 880	72 090	73 300	74 510	75 720	76 930
60 000	61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000
65 500	66 300	67 100	67 900	68 700	69 500	69 300	71 100	71 900	72 700	73 500	74 300	75 100	75 900
61 380	62 330	63 280	64 230	65 180	66 130	67 080	68 030	68 980	69 930	70 880	71 830	72 780	73 730
62 090	63 300	64 510	65 720	66 930	68 140	69 350	70 560	71 770	72 980	74 190	75 400	76 610	77 820
61 000	62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000	74 000
67 000	67 800	68 600	69 400	70 200	71 000	71 800	72 600	73 400	74 200	75 000	75 800	76 600	77 400
62 430	63 380	64 330	65 280	66 230	67 180	68 130	69 080	70 030	70 980	71 930	72 880	73 830	74 780
62 980	64 190	65 400	66 610	67 820	69 030	70 240	71 450	72 660	73 870	75 080	76 290	77 500	78 710
62 000	63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000	74 000	75 000
68 500	69 300	70 100	70 900	71 700	72 500	72 300	74 100	74 900	75 700	76 500	77 300	78 100	78 900
63 480	64 430	65 380	66 330	67 280	68 230	69 180	70 130	71 080	72 030	72 980	73 930	74 880	75 830
63 870	65 080	66 290	67 500	68 710	69 920	71 130	72 340	73 550	74 760	75 970	77 180	78 390	79 600
63 000	64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000	74 000	75 000	76 000
70 000	70 800	71 600	72 400	73 200	74 000	74 800	75 600	76 400	77 200	78 000	78 800	79 600	80 400
64 530	65 480	66 430	67 380	68 330	69 280	70 230	71 180	72 130	73 080	74 030	74 980	75 930	76 880
64 760	65 970	67 180	68 390	69 600	70 810	72 020	73 230	74 440	75 650	76 860	78 070	79 280	80 490
64 000	65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000	74 000	75 000	76 000	77 000
71 500	72 300	73 100	73 900	74 700	75 500	76 300	77 100	77 900	78 700	79 500	80 300	81 100	81 900
65 580	66 530	67 480	68 430	69 380	70 330	71 280	72 230	73 180	74 130	75 080	76 030	76 980	77 930
65 650	66 860	68 070	69 280	70 490	71 700	72 910	74 120	75 330	76 540	77 750	78 960	80 170	81 380
65 000	66 000	67 000	68 000	69 000	70 000	71 000	72 000	73 000	74 000	75 000	76 000	77 000	78 000
150	150	150	150	150	150	150	150	150	150	150	150	150	150
105	105	105	105	105	105	105	105	105	105	105	105	105	105
89	89	89	89	89	89	89	89	89	89	89	89	89	89
100	100	100	100	100	100	100	110	100	100	100	100	100	100

Mr. Webster

element will be represented by a curve. This may be an arc of a circle, or a parabola, or a cycloid, or a broken line; it may be different in degree or different in nature in the case of each element; and it may vary in degree or even in nature with changes in the proportions of the associated elements. But it will be assumed in this investigation that, within the narrow limits of the divisions of the table, the effect of a regular increase in the percentage of each metalloid would be represented by a straight line. In other words, that an increase of carbon from .20 to .21% gives the same increment in strength as an increase from .10 to .11 per cent."

In order to study more closely the relations between the estimated ultimate strengths arrived at by the writer's method and those of Messrs. Campbell and Cunningham, the tables have been constructed on the same general plan as Table No. 5, giving the values of all three observers on each table, the carbons being in all cases from .06 to .28% inclusive. The values are for open-hearth steel, and sulphur has not been considered, but it can be used in connection with these tables by adding 500 lbs. for each .01% of sulphur above .065% sulphur, and deducting from the values given 500 lbs. for each point below this amount. In Table No. 5* the manganese is zero in all cases, and for that reason the values for Mr. Campbell's acid steel and Mr. Cunningham's values are higher than the others, as in these cases the value of manganese has been indirectly included in that of carbon. Taking, however, Tables Nos. 6, 7, 8 and 9,* with manganese 30, .40, .50 and .60, respectively, the estimated ultimate strengths of one observer agree with those of the other much more closely than would be expected from the different values used for each element. Tables Nos. 10 to 16*, inclusive, are for carbons; 29 to 51, inclusive, with manganese .0, .50, .60, .70, .80, .90, and 1 per cent. In all other respects they are the same as Tables Nos. 5 to 9. Of course, with the higher carbons these values may not apply, but the sooner they are tried, the sooner will the required modifications be known.

The writer believes that no one has as yet arrived at the right values for the different elements, and doubts if we will ever have a simple mathematical formula to express these values, as the elements present have, no doubt, an effect upon each other. It will, of course, take much time and work to decide on the best modifications of values and methods now in use.

The writer has quoted freely from a paper written for presentation before the American Institute of Mining Engineers, which gives a general summary of the work of many investigators on this problem, and is a more complete answer to some of the points raised in this discussion.

* These tables, 5 to 16 inclusive, are filed in the Library of the Society for reference. Table No. 7 is printed with this discussion (see pages 468 and 469 in order to show the comparison referred to.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

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No. 841.

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH.

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PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

WITH DISCUSSION.

Through the courtesy of C. K. Bannister, M. Am. Soc. C. E., and at his suggestion, the authors were enabled to carry out, in August, 1897, a limited series of experiments on the flow of water in the recently finished conduit of the Pioneer Electric Power Company, of Ogden, Utah, of which he was at that time chief engineer. As the main dam has not been constructed, it was not possible to extend the experiments to the velocities of flow corresponding to the full carrying capacity of the conduit. It is the intention to do so when opportunity offers. Believing that the results obtained so far, and a description of the method of experimentation adopted, may prove of interest to the profession, this paper is submitted.

For a description of the entire pipe line and of the construction of the pipe, reference may be made to a paper recently published.* The plan and profile of each of the portions experimented upon are given herewith, in connection with the account of the experimental results.

I.—Description of Object and Methods.

General Plan.

The main object of the experiments was to determine the relation between the rate of discharge of the pipe and the loss of head between certain definite points. To accomplish this it was necessary to measure, simultaneously, the pressure at each end of the length of pipe under experiment, and the rate of discharge.

As a secondary object, observations were taken on the loss of head in the Venturi meters, with different velocities of flow.

Pressure Measurements.

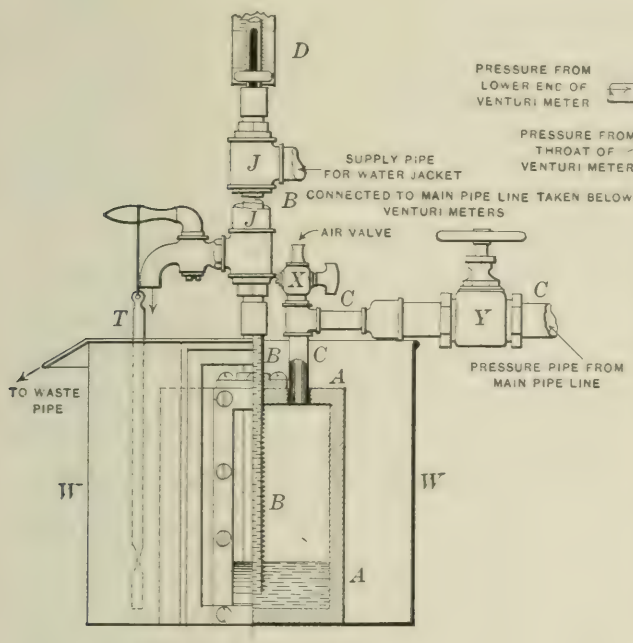
(*1*) *Method Adopted.*—A consideration of the probable range of the losses of head to be measured made it evident that a far greater degree of precision was required in the pressure measurements than could be attained with any pressure gauge of the Bourdon type. It seemed reasonably certain that the best results attainable with such a gauge would be unreliable by at least as much as 1 lb. per square inch (equivalent to a water column of 2.3 ft.); more probably the errors would be two or three times that amount, or even greater. Such a degree of uncertainty would render the results wholly valueless. It was therefore decided to use mercury gauges.†

(*2*) *Description of Gauges.*—The form of gauge used was the ordinary open manometer. Fig. 1*a* shows the essential parts of the manometer as used at the point of greatest pressure, near the power house. Besides the manometer proper, the figure shows a water-jacket surrounding the mercury tube, and a vessel into which the water passing

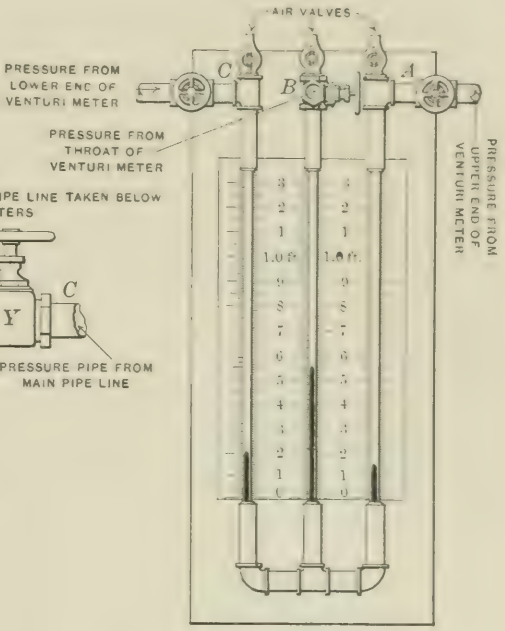
* "The Power Plant, Pipe Line and Dam of the Pioneer Electric Power Company at Ogden, Utah," by Henry Goldmark, M. Am. Soc. C. E. *Transactions*, Vol. xxxviii, p. 246.

† This discussion of Bourdon gauges has reference only to their applicability to this series of experiments. It is not meant to imply that such gauges can never give satisfactory results in experiments of this kind. For a great length of pipe the loss of head may be so great that the errors of a Bourdon gauge are relatively unimportant. In any case, however, such gauges should be repeatedly and carefully tested, during the period of experimenting, at about the pressures at which they are used. It will aid in eliminating gauge errors if the pressure is measured at various points along the profile of the pipe.

through the water-jacket wasted; the mercury reservoir, being placed in this vessel and being surrounded by water. The mercury reservoir A is of cast iron, the interior being cylindrical, 4 ins. in diameter. In one side is a window, fitted with glass, through which the position of the mercury surface may be observed. The vertical tube $B\,B$ is of ¼-in. wrought-iron pipe, except at the upper end, where a length of glass tube D is attached, with a scale for reading the position of the top of the mercury column. Into the top of the reservoir A is fitted a pipe $C\,C$, leading to that cross-section of the main pipe at which the pressure is to be measured. In the pipe $C\,C$ is a gate-valve Y, and at



LOWER MANOMETER
FIG. 1*a*.



DIFFERENCE GAUGE
FIG. 1*b*.

X is an air-cock. When the gauge is in use the pipe $C\,C$ and the space in the reservoir above the mercury are filled with water. The air in the reservoir is removed by opening the air-cock X and allowing water to enter slowly through the partially open valve Y. The air-cock is then closed and the valve Y fully opened, and mercury rises in the tube $B\,B$. The height of the mercury column measures the water pressure at the surface of the mercury in the reservoir; and thus, by allowing for difference of level, the pressure is known at any point of the pipe $C\,C$ and of the cross-section of the main pipe to which this pipe is attached.

Two questions in connection with the use of the gauge call for some explanation—the effect of changes of temperature upon the mercury column, and the method of attachment of the gauge to the main pipe.

(3) *Correction for Temperature.*—In order that the observations may be comparable, the observed height of the mercury column must in every case be reduced to that of an equivalent column at some standard temperature. It was therefore necessary to know the temperature of the mercury at the time of each observation of its height. The degree of precision needed in the temperature observations may be estimated by noticing that a change of 1° Cent. affects the length of the mercury column by about $\frac{1}{5550}$ of its value. The static pressures at the four points at which measurements were made (expressed in height of water column) were about 462 ft., 153 ft., 101 ft. and 48 ft., respectively. Temperature changes of about 1.2°, 3.6°, 5.5° and 11.5° Cent. at these four points, respectively, would thus each correspond to an error of 0.1 ft. in the measured pressure-head.

At the lower end of the steel pipe, where the pressure had its greatest value, it was decided to use a water-jacket for the purpose of keeping the mercury at a nearly constant temperature. This jacket is shown in Fig. 1*a*, already referred to. The $\frac{1}{4}$-in. mercury tube was enclosed in a larger pipe *J J*, through which a stream of water was allowed to flow continuously during the time of experimenting. The temperature of this water remained nearly constant, and was read at intervals by means of a chemical thermometer *T*, suspended in the waste-vessel *W*. This water-jacket surrounded a length of about 30 ft. of the column, the total length of which varied from 34 ft. to 32.8 ft. A length not exceeding 4 ft. was thus left unjacketed; but it is believed that no serious error was thus introduced. Thus, for a length of 4 ft. of mercury, an error of about 10° Cent. would be required to introduce an error of 0.1 ft. (water) into the pressure-head.

At the other pressure stations the heights of the mercury columns were much less, and it was thought sufficient to determine the temperature by means of a thermometer placed beside the manometer tube. During most of the observations taken at the upper end of the steel pipe line, where the greatest height of the mercury was about 11.3 ft., the instrument was shaded during the time of experimenting. At the upper station on the wooden pipe line the manometer was fully

shielded from the sun by a shed built for the protection of the pipe against rock slides, and the temperature of the air was very constant. At the lower station on the wooden pipe line the tube containing the mercury column was shielded from the sun by a 2-in. plank upon which it was mounted, and the temperature was read from a thermometer hanging beside it.

Although, in some respects, more favorable temperature conditions would have been desirable at two of the manometer stations, it is not believed that serious error was introduced into the results from this cause.

(*4*) *Attachment of Manometers to Main Pipe.*—The proper method of attachment of a piezometer for the purpose of measuring the pressure at any section of a pipe carrying water has been the subject of considerable discussion. The main questions raised have been: (*a*) whether a single point of attachment to the pipe is sufficient, or whether attachment should be made to a chamber communicating with the pipe at several points of the circumference; and (*b*) whether in case attachment is made only at a single point, this should be at the top or at some other point on the circumference. A careful consideration of the principles involved led the authors to the conclusion that the number of points of attachment is immaterial, and that the position of the point of attachment on the circumference is important only as affecting the liability to the collection of air in the pipe leading to the piezometer.*

In the case of the steel pipe the simplest method available was to make connection with relief valves at the top of the pipe. At the lower end the manometer was located in the power house, about 200 ft. from the point of attachment to the main pipe; the connecting pipe, $\frac{3}{4}$ in. in diameter, entering the building through a window. At the highest point of this connecting pipe was placed a valve for drawing off air. At the station near the upper end of the steel pipe line the connection with the manometer was made through a pipe about 20 ft. in length, with a valve for air at the highest point. In both cases the air valves were opened at intervals during each run, in order to remove any possible accumulation of air.

In connecting the manometer to the wooden pipe, a hole was bored at right angles to the axis of the pipe. Into this hole the pipe lead-

* A further discussion of these questions is given in the appendix to this paper.

ing to the manometer was screwed, care being taken that there should be no projection beyond the inner surface. At each of the pressure stations on the wooden pipe line the length of the connecting pipe was only a few feet.

On Plate XXXVII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the wooden pipe; and on Plate XXXVIII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the steel pipe. The difference-gauges on the north and south Venturi meters, respectively, are shown by Figs. 1 and 2, Plate XXXIX.

Measurement of Rate of Discharge.

(*1*) *Method Adopted.*—The main pipe, 72 ins. in diameter, divides near the power house into two branches, each 54 ins. in diameter. Each of these branch pipes is furnished with a Venturi meter with automatic register. These meters were the only available means of measuring the rate of discharge with any approach to precision.

The automatic registers, however, were not adapted to the purpose in hand. The register is an integrating device, recording the total quantity discharged, but not showing the rate of discharge directly. The rate of discharge can be determined from its readings only by observing the difference between the readings at the beginning and end of a known period of time. This period must be of considerable length, since the smallest quantity directly indicated by the register is 10 000 cu. ft.; and intermediate readings cannot be estimated with precision. Moreover, the result of such a determination is not reliable unless the rate of discharge has remained uniform, since the indicating mechanism moves only once in ten minutes, registering at each movement an amount proportional to the rate of discharge at that instant. The register itself gives no indication as to what fluctuations in the rate of discharge occur during the ten-minute intervals. It was therefore decided to attach to each meter a mercury gauge, designed to show, at every instant, the difference between the pressures existing at the throat and the up-stream sections of the Venturi. This pressure difference indicates the rate of discharge instantaneously; no time interval need be measured.

(*2*) *Difference-Gauges.*—For accomplishing the object stated, the essential parts of the difference-gauge are two vertical glass tubes

FIG. 2.

FIG. 1.

connected at the bottom, and at the top communicating respectively with the two sections whose pressure difference is to be determined. The tubes are filled with mercury to a convenient height, the space above the mercury and the connecting pipes being filled with water. By means of a fixed vertical scale, the difference in level between the tops of the two mercury columns may be read at any desired instant.

The gauges, as actually constructed, were designed to accomplish an additional object, namely, to indicate the total loss of head in the meters. For this purpose a third tube was added, connected with the other two at the lower end, and at the top communicating with the 54-in. pipe immediately below the meter. Simultaneous readings of the three mercury columns thus served to determine both (*a*) the instantaneous rate of discharge, and (*b*) the corresponding loss of head in the meter.

In Fig. 1*b* are shown the essential features of these difference-gauges as actually constructed and used. The three glass tubes communicate with each other freely at the bottom through suitable connections of wrought-iron pipe. At the top they communicate with separate pipes *A*, *B*, *C*, leading respectively to the three sections whose pressures are to be compared. Each is furnished at the highest point with an escape cock for drawing off air, and in each pipe is placed a gate-valve.

In attaching the gauge for use, the air-valves were at first left open, and the space in the tubes above the mercury completely filled with water, the gate-valves in the connecting pipes being, for this purpose, only slightly opened. These latter valves were tightly closed as soon as all the air had escaped from the tubes and connecting pipes, and very gradually opened again after the air-valves had been closed. No difficulty was experienced from air in the pipes *A*, *B*, *C*. In two of these pipes (*A* and *B* in each gauge) the air-valve was at the highest point; in the third (that running to the section below the meter) this was not the case. Whether air was present in any pipe could be tested at times of no discharge, since under static conditions the three mercury columns must stand at the same level if the connecting pipes were completely filled with water. During the series of experiments there were frequent opportunities of applying this test, and air was never found to be present, even in the pipe with the upward bend.

except in two or three instances. These instances occurred after the adjacent receiving chamber had for some reason been emptied of water; and it was always found easy to blow off the air through the air-valve V.

The scales for reading the heights of the mercury in the tubes were graduated to hundredths of a foot, and the third decimal place was estimated in taking readings. In setting up the instrument, the adjustment was determined by the equality of the readings of the three scales under static conditions.

Programme of Tests.

In carrying out the experiments, an observer stationed at each of the gauges in use took readings at short intervals during a certain time previously agreed on. The interval between readings was originally intended to be one minute, but it was found that under steady conditions it was not always necessary to read so frequently.

The rate of discharge was controlled by the superintendent in charge of the power house. The aim was to maintain a uniform flow for a certain period; then to change the rate of discharge and maintain a uniform flow for another period; then change again; and so on as long as desired. In this way was obtained a series of simultaneous sets of values of the three quantities sought, the rate of discharge and the pressures at two sections of the main pipe.

It was not necessary to note the time of reading with great precision, so long as nearly uniform conditions of discharge and pressure could be maintained. A sufficient degree of uniformity was secured without difficulty, except in two or three instances to be mentioned later.

After a change in the rate of discharge, some time always elapsed before a steady condition was re-established. It was found that about fifteen minutes must be allowed for each change. The first day's results were unsatisfactory because sufficient time was not allowed for steady conditions to be re-established after changes in the rate of discharge. During the period of adjustment from one steady condition to another, the pressure gauges showed marked fluctuations, the range being at first considerable,* but decreasing and nearly disappearing

*Although no accurate determination was made of the increase of pressure caused by sudden stoppage or change of the rate of discharge, it is certain that, even at the lower end of the line, the mercury column in no case rose more than 3 ft. above the static height. In one case a very sudden stoppage occurred, owing to the blowing out of a fuse, but the above statement holds even for this extreme case.

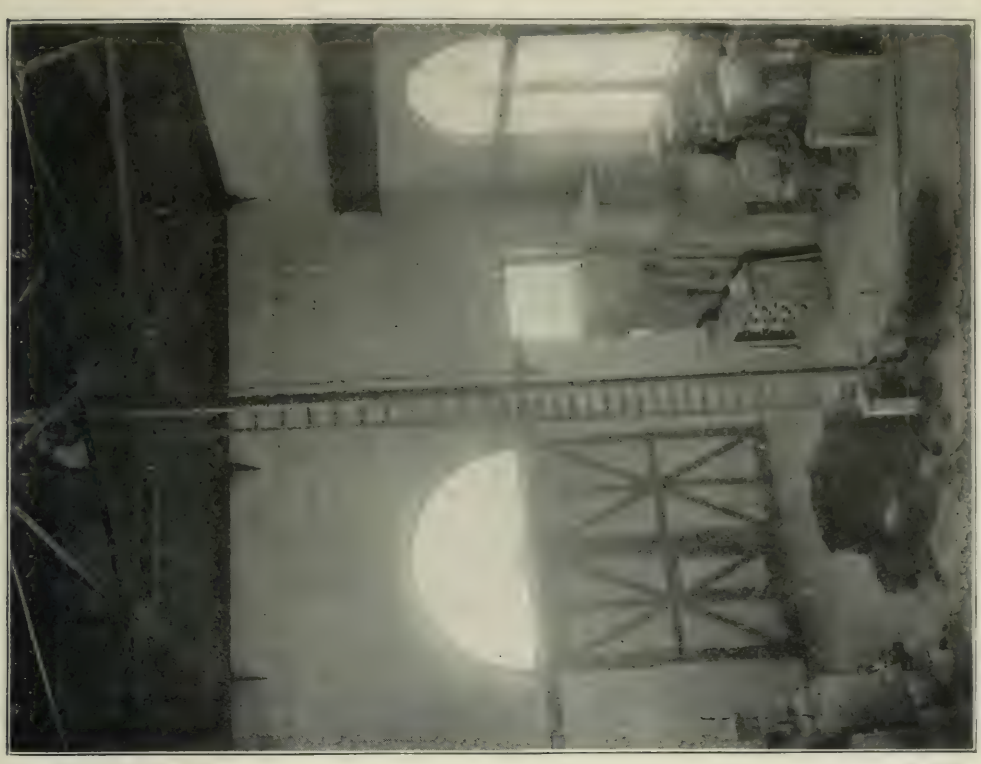


FIG. 2.

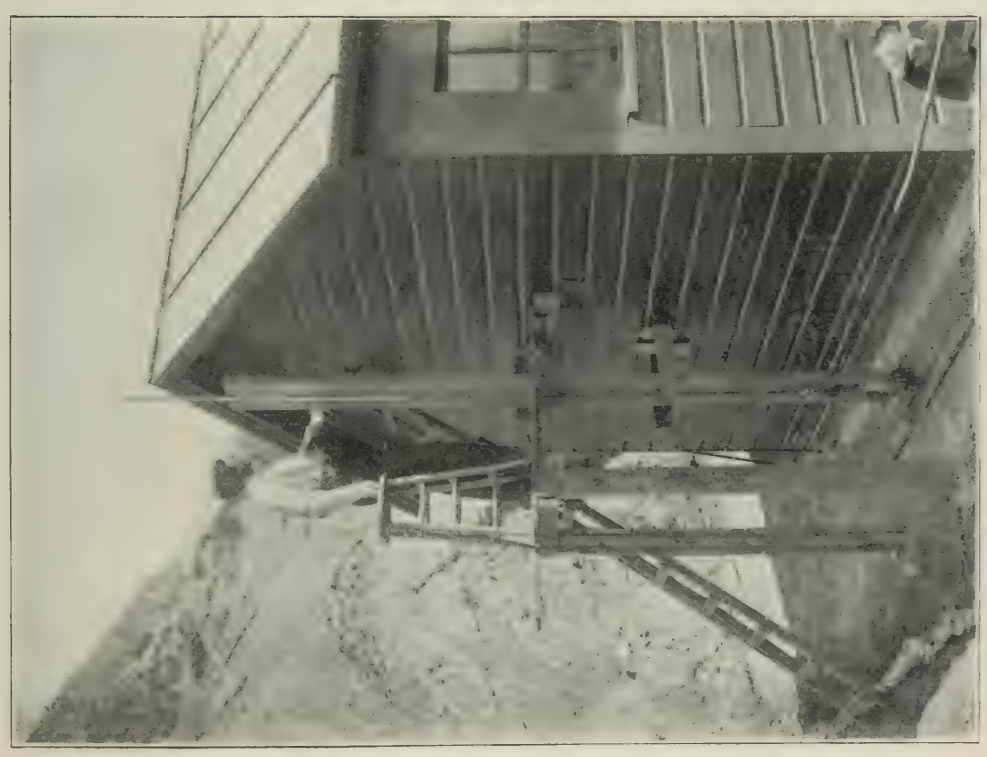


FIG. 1.

after ten or fifteen minutes. The difference-gauges attached to the Venturi meters, on the other hand, adjusted themselves to changed conditions much more quickly, their oscillations at times of change continuing not more than two or three minutes. This showed that the rate of discharge became steady very quickly, while the pressure continued to oscillate for some time. It was at first suggested that the observed oscillations of the manometer columns might be due to the inertia of the mercury. It was found, however, by plotting curves showing the simultaneous readings of the two gauges in use at the same time, that there was always a close agreement between the periods of oscillation of the two columns. This is shown in Fig. 5. It would therefore appear that the observed oscillations correspond to true oscillations of pressure in the pipe. Doubtless these variations of pressure are transmitted in waves throughout the length of the pipe. No difference of phase between the oscillations of pressure at the two stations could be detected.*

Because of conditions existing at the inlet, the highest rates of discharge observed were accompanied by a decrease of pressure throughout the pipe. Under such circumstances it was of more importance to secure exact coincidence in the times of observing the two pressures, since any uncertainty as to what observations at the two stations are simultaneous introduces an uncertainty into the value of the loss of head between the stations. It was found that the gauges at the two stations always gave practically identical determinations of the rate of decrease of the pressure. This is illustrated by the plotted results shown in Fig. 4.

It will be noticed that the oscillations of pressure at the beginning of the period represented by the curves in Fig. 5 show a period between successive maxima or minima of about six minutes. The records for the entire series of experiments show a period lying between five and six minutes. It should be said that there were also minor oscillations of a period so short that no attempt was made to record them. The period of these minor oscillations was not determined, but it was only a few seconds.

In cases of falling pressure the oscillations recorded at the two stations serve as a check upon the coincidence of the instants of observation.

* The velocity of propagation of a pressure wave should, in fact, be practically equal to the velocity of sound in water—about 4 700 ft. per second. The interval occupied in traversing the distance between the two manometers would therefore be less than one second, since the greatest length of pipe experimented upon was 4 427 ft.

Reduction of Observations.

(*1*) *Reduced Height of Manometer Column.*—The quantity directly given by a manometer reading was the height of the top of the mercury column above the zero of the scale. To this must be added the height of the zero of the scale above the surface of the mercury in the reservoir. This latter quantity need not be determined with great accuracy, since by the method adopted for computing loss of head (to be explained presently) a constant added to all the observed readings does not affect the final result. Except for the necessity of applying the temperature correction, the direct readings of the scale could be used instead of the total heights of the mercury column. For the purpose of applying the corrections for temperature, the height of the zero of the scale above the surface of the mercury in the reservoir was taken as a constant, its value being measured under conditions of static pressure. The reduction to an equivalent column at 0° Cent. is made by multiplying the observed height of the column by the factor $1-0.000\,18\,T$; T being the temperature of the mercury at the time of observation, expressed in degrees Centigrade.

Strictly speaking, a correction must also be made for the rise of the surface of the mercury in the reservoir. The interior diameter of the reservoir being about sixteen times that of the glass tube, a fall of the mercury in the latter must be accompanied by a rise about $\frac{1}{256}$ as great in the former. The true shortening of the column is therefore equal to the drop of the column multiplied by $\frac{257}{256}$, or about 1.004. For the greatest loss of head measured in the entire series of experiments (about 3.4 ft.), this correction would amount to only 0.014 ft. (water).

(*2*) *Plotting of Reduced Readings.*—After each day's run, the reduced heights of the manometer columns and the readings of the Venturi difference-gauges were plotted as ordinates with time as abscissas. By inspection of the resulting curves it was possible to select periods during which the conditions of pressure and discharge remained steady. The data corresponding to each period were averaged, and the resulting average values of the manometer heights and difference-gauge readings were regarded as constituting an "observation." Samples of the plotted results are shown in Figs. 4 and 5 which will be referred to again.

FIG. 2.

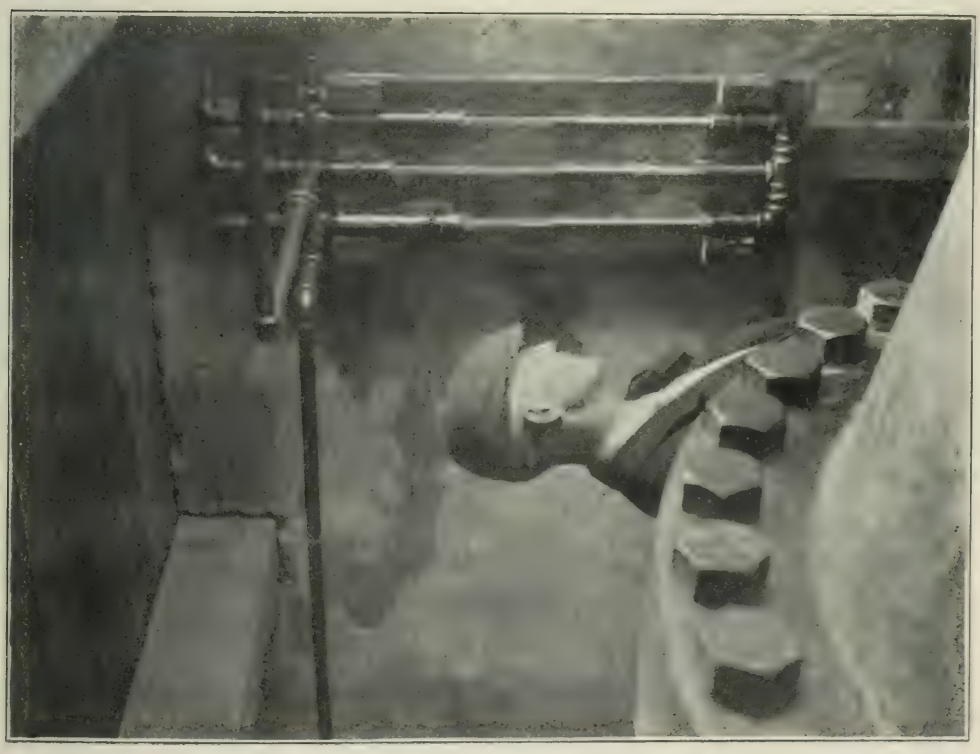


FIG 1.

(*3*) *Computation of Loss of Head.*—Since the pipe had the same diameter at the two pressure stations, the loss of head between these stations is found by a comparison of the pressures.

Let suffixes $(_1)$ and $(_2)$ refer to up-stream and down-stream stations respectively, and let

m_1 and m_2 denote any simultaneous heights of the mercury columns ;

M_1 and M_2 values of m_1 and m_2 under static pressure ;

$y = m_1 - m_2$;

$Y = M_1 - M_2$;

e = specific gravity of mercury = 13.6 ;

H' = loss of head between the two stations (expressed in height of water column).

Then $e\,(M_1 - m_1)$ = loss of head in pipe from inlet to upper station ;

$e\,(M_2 - m_2)$ = " " " " lower "

$e\,[\,(M_2 - m_2) - (M_1 - m_1)\,] = e\,(y - Y)$ = loss of head between the two stations..... (1)

That is, $H' = e\,(y - Y)$..... (2)

It is evident that the value of H' is not changed by any constant error affecting all values of m_1 (including the static value), nor by a constant error affecting all values of m_2. This justifies the statement above made that great accuracy is not required in the determination of the height of the zero of the manometer scale above the surface of the mercury in the reservoir. Any constant length of the mercury column could be disregarded, except for the necessity of reducing for temperature.

(*4*) *Computation of Rate of Discharge.*—The theory of the Venturi meter is based upon the assumption that the velocity of flow is uniform throughout each of the cross-sections whose pressures are compared. On this assumption, and neglecting loss of head between the two sections, the rate of discharge is given by the following formula :

Let Q denote the rate of discharge in cubic feet per second, computed on the above assumption, and let the difference between the pressure-heads at points on the same level in the two cross-sections (called by Herschel "head on Venturi") be represented by H (expressed in feet of water column). Then, if a_1 and a_2 are the areas of the two cross-sections,

$$Q = \frac{a_1\, a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2\,g\,H} \qquad \ldots\ldots (3)$$

If q denotes the true value of the rate of discharge,

$$q = kQ \ldots\ldots (4)$$

k being a coefficient whose value is known only by experiment.

In the Ogden meters the diameters of the two cross-sections are 54 ins. and 25.5 ins., respectively. The formula therefore becomes

$$q = 3.639\, k\sqrt{2\,g\,H}; \ldots\ldots (5)$$

q being in cubic feet per second, H in feet, g in foot-second units, and k being an abstract number.

In order to save labor in the application of the formula it is convenient to replace H by the difference in height of the two mercury columns of the difference-gauge. If h denotes this difference and e the specific gravity of mercury,

$$H = (e - 1)h; \ldots\ldots (6)$$

since the difference between the mercury heights is in part balanced by an equal and opposite difference between the heights of the water columns above the mercury. Taking 13.6 as the value of e, equation (5) may be written

$$q = 103.58\, k\sqrt{h} \ldots\ldots (7)$$

The values of k to be used with the Ogden meters were furnished to the writers, at the request of Clemens Herschel, M. Am. Soc. C. E., by the manufacturers, the Builders' Iron Foundry, Providence, R. I.

If

$$k^1 = \frac{a_1}{\sqrt{a_1^2 - a_2^2}}\, k \ldots\ldots (8)$$

the values of k^1 are given in the following table:

Head on Venturi. H (feet).	Coefficient. k^1.	Head on Venturi. H (feet).	Coefficient. k^1.
1	.995	13	.977
2	.992	14	.9765
3	.9895	15	.9755
4	.9875	16	.975
5	.986	17	.974
6	.985	18	.973
7	.9835	19	.9725
8	.9825	20	.9715
9	.9815	21	.971
10	.9805	22	.9705
11	.9795	23	.970
12	.9785		

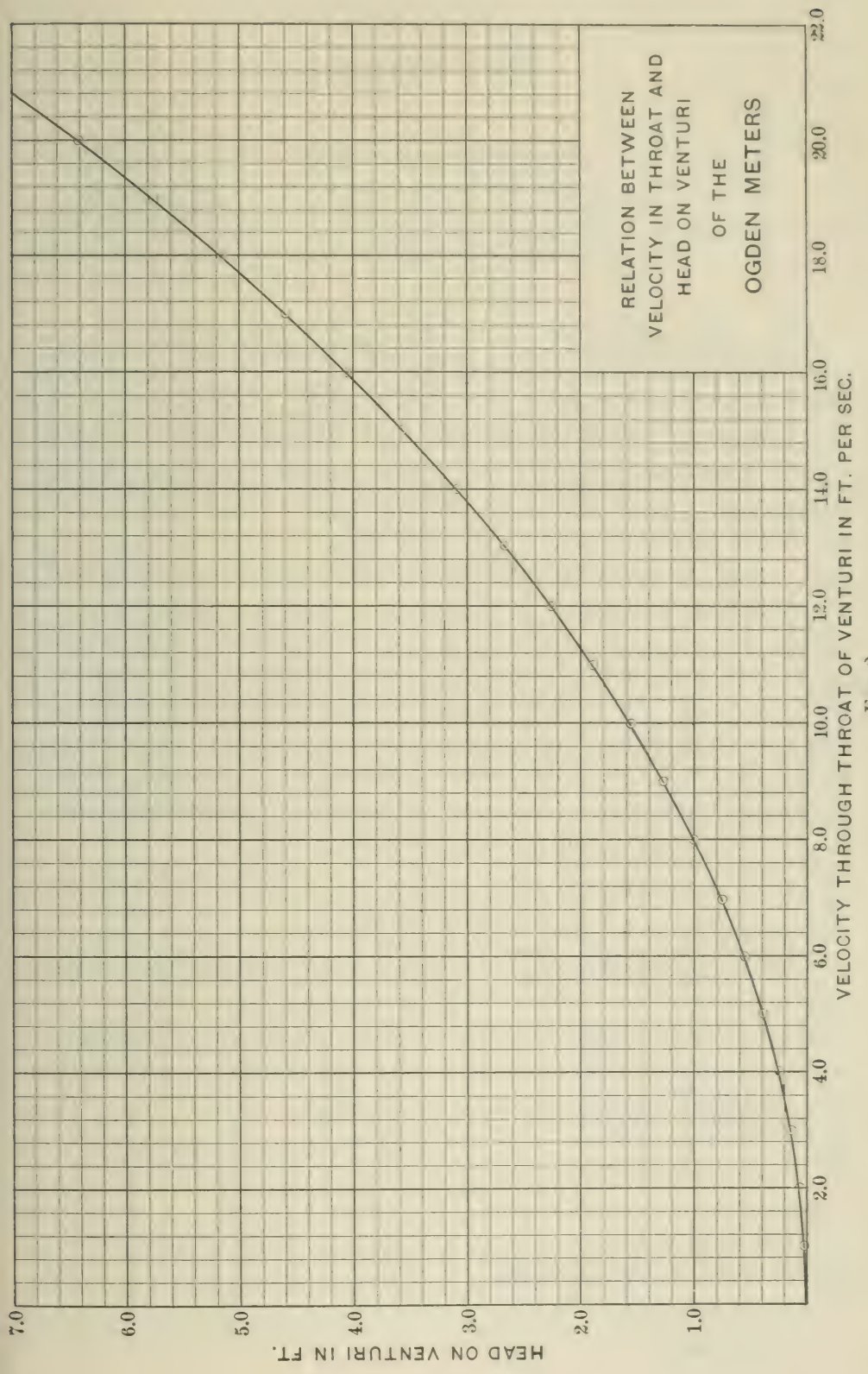

RELATION BETWEEN
VELOCITY IN THROAT AND
HEAD ON VENTURI
OF THE
OGDEN METERS
HEAD ON VENTURI IN FT.
7.0
6.0
5.0
4.0
3.0
2.0
1.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
VELOCITY THROUGH THROAT OF VENTURI IN FT. PER SEC.

FIG. 2.

These values are based upon experiments made by Herschel in 1891 with the 48-in. meter of the East Jersey Water Company; the ratio of throat diameter to main diameter being the same for this meter as for the Ogden meters.

The values of the throat velocity for different values of "head on Venturi," corresponding to the above values of the coefficient k^1, are shown in Fig. 2. This diagram is copied from one furnished by the Builders' Iron Foundry, which was used in making the actual computations.

The heights of the mercury columns of the difference-gauges were read to 0.001 ft., and it is believed that the probable error in a value of h does not exceed 0.002 ft. The corresponding error in the value of q varies with q, being less as q is greater. From the relation between q and h above given, taking $k = 1$ (which is sufficiently correct for the present purpose),

$$d\,q = \frac{5360}{q}\,d\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

An error $\delta\,h = 0.002$ causes an error $\delta\,q = \frac{10.7}{q}$, which varies from 1.1 to 0.11 as q varies from 10 cu. ft. per second to 100 cu. ft. per second. The smallest values of q in the experiments under consideration were about 14 cu. ft. per second; the uncertainty of these values, so far as due to errors in the observed values of h, is probably about 5 per cent. The observational error in values of q greater than 35 cu. ft. per second is probably less than 1 per cent.

General Results.

In order to facilitate comparison with the results of other experiments, the results are represented in three ways:

(1) The loss of head per 1 000 ft. has been computed from each observation, and the relation between this loss and the velocity of flow represented graphically.

(2) The values of c in the Chezy formula,

$$v = c\sqrt{r\,s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

has been computed for each observation, and the relation between v and c represented graphically.

(3) The value of f in the formula for loss of head in a pipe of diameter d and length l,

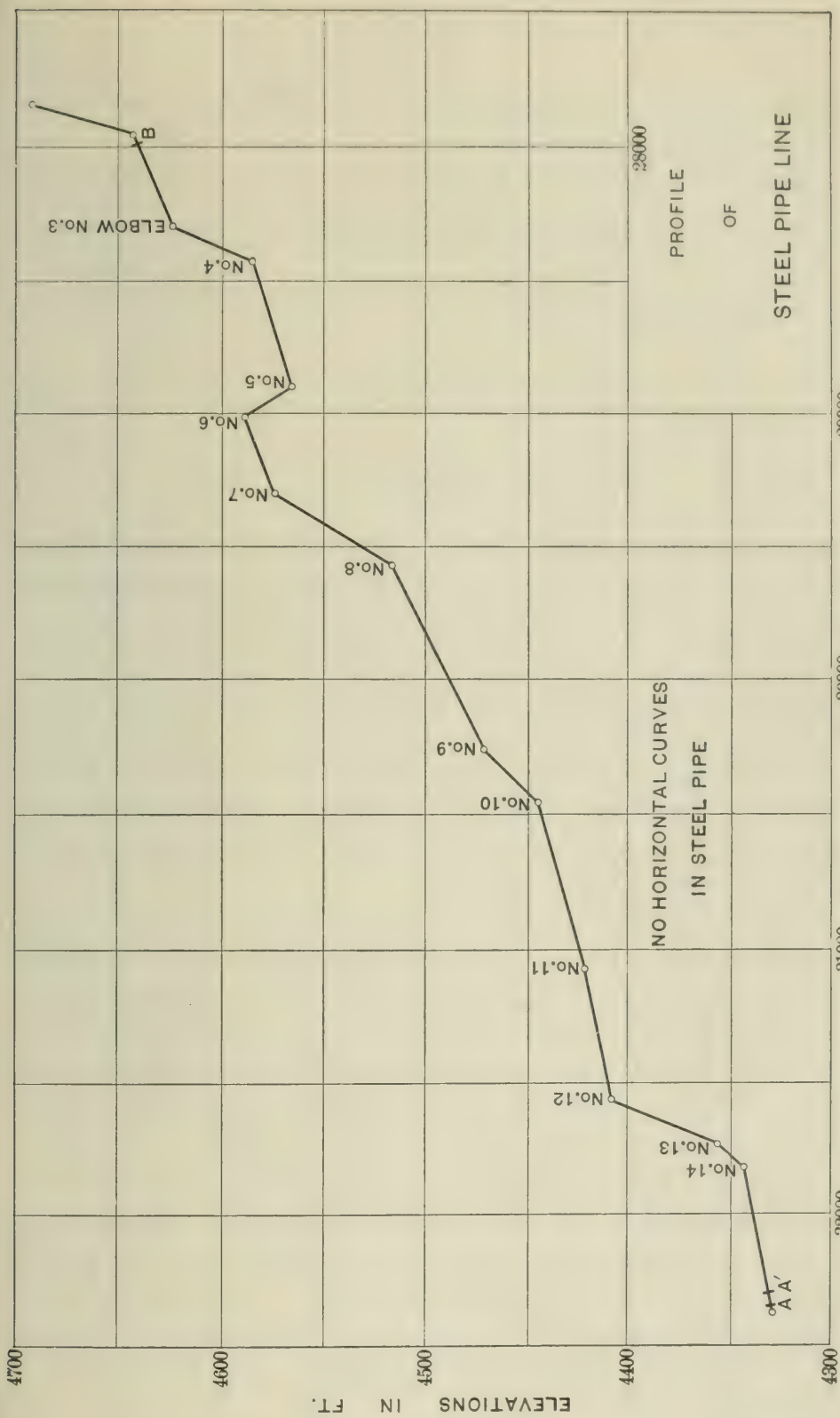

PROFILE
OF
STEEL PIPE LINE
NO HORIZONTAL CURVES
IN STEEL PIPE
ELEVATIONS IN FT.
HORIZONTAL DISTANCES IN FT.
4700
4600
4500
4400
4300
28000
29000
30000
31000
32000
A
A'
B
ELBOW No.3
No.4
No.5
No.6
No.7
No.8
No.9
No.10
No.11
No.12
No.13
No.14

FIG. 3.

$$H' = f \frac{l}{d} \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

has been computed for each observation, and the relation between f and v represented by a diagram.

There is a definite relation between c and f. In the Chezy formula the value of s, the hydraulic slope, is $\frac{H'}{l}$ and the value of the hydraulic radius r is $\frac{d}{4}$. The above two formulas are therefore identical if

$$f = \frac{8g}{c^2} \quad \ldots\ldots,\ldots\ldots\ldots\ldots \quad (12)$$

Loss of Head in Venturi Meter.

As already stated, the difference-gauges attached to the Venturi meters were designed to show the differences between the pressures at three sections: (1) just before the contraction of the stream, the diameter being 54 ins.; (2) at the throat, the diameter being 25.5 ins., and (3) just below the expanding stream, the diameter being 54 ins. The difference between the pressures at the first and third sections shows the loss of head caused by friction, and by the contraction and expansion of the stream in passing through the meter. The relation between this loss and the rate of flow through the meter is represented both graphically and in tabular form.

Some trouble was experienced from the presence of dirt at the top of the mercury columns of the difference-gauges. It fortunately happened that this difficulty was confined, in both gauges, to the tube communicating with the section below the meter (the tube marked *C* in Fig. 1*b*), so that the estimate of rate of discharge was not affected. The observations for loss of head in the meter were, however, rendered less accurate than they would otherwise have been. The uncertainty in the reading of the mercury column probably in no case exceeded 0.004 ft., but this was sufficient to render valueless certain of the observations taken with low velocities of flow. In spite of this, the entire series of observations serves to show pretty satisfactorily the relation between loss of head in meter and velocity of flow, within the range of the experiments. The discussion of results follows.

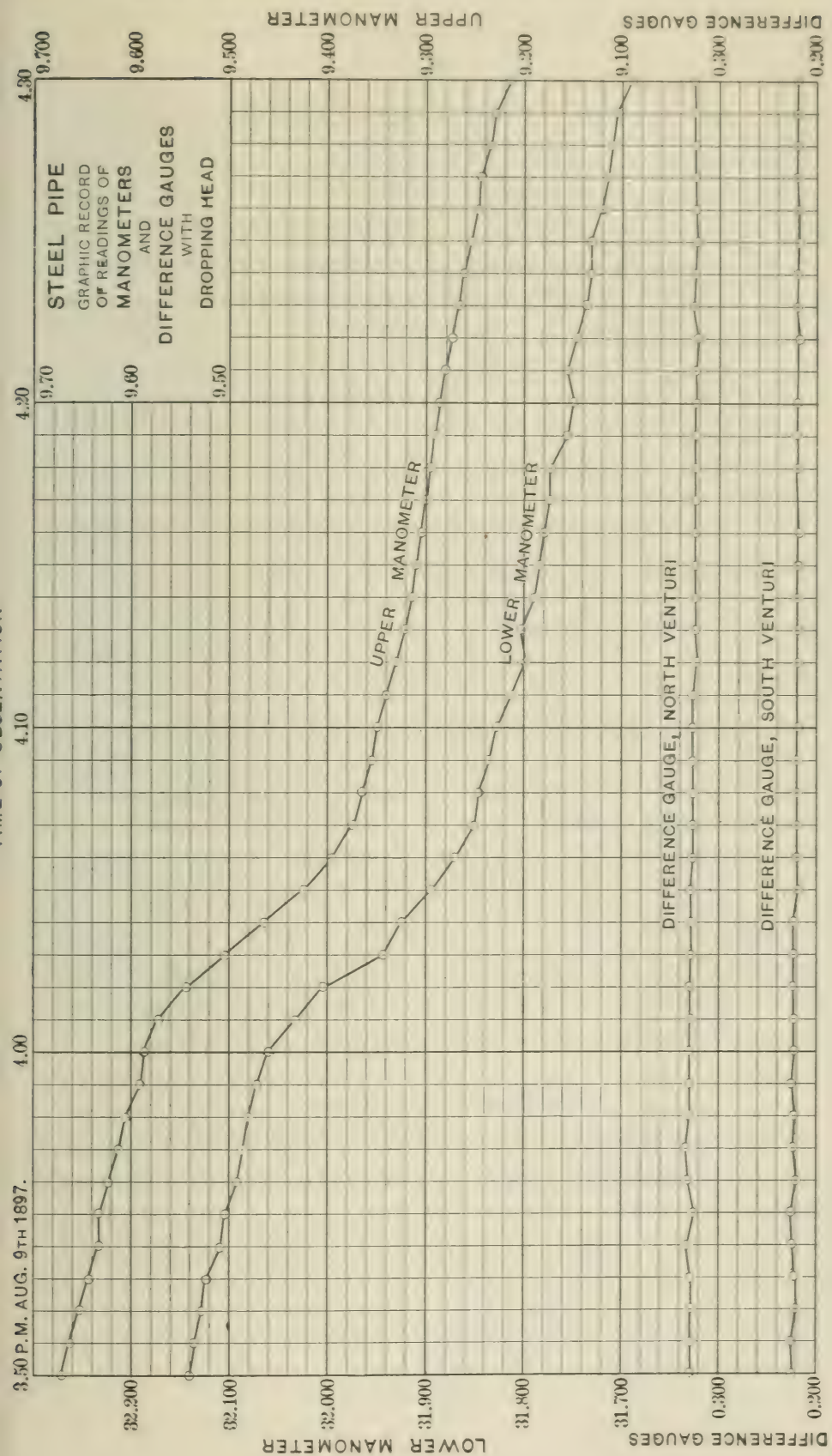

STEEL PIPE
GRAPHIC RECORD
OF READINGS OF
MANOMETERS
AND
DIFFERENCE GAUGES
WITH
DROPPING HEAD
3.50 P.M. AUG. 9TH 1897.
4.00
4.10
4.20
4.30
9.70
9.60
9.50
9.700
9.600
9.500
9.400
9.300
9.200
9.100
0.300
0.200
32.200
32.100
32.000
31.900
31.800
31.700
UPPER MANOMETER
LOWER MANOMETER
DIFFERENCE GAUGE, NORTH VENTURI
DIFFERENCE GAUGE, SOUTH VENTURI
LOWER MANOMETER
UPPER MANOMETER
DIFFERENCE GAUGES
DIFFERENCE GAUGES

FIG. 4.

II.—Experiments on Steel Pipe.

Description of Pipe Line.

For a full description of the dimensions and construction of the riveted steel pipe, reference may be made to the paper previously cited.* The portion experimented upon included nearly the entire length of the steel pipe. In plan this pipe is straight; the profile is shown in Fig. 3. The positions of vertical curves or elbows are also shown in Fig. 3, the radius of curvature and total angle in each case being given in the following table:

Elbow No.	Station.	Angle.	Radius in feet.
3	283	13° 30′	30
4	284 + 80	14° 23′	30
5	289	14° 8′	30
6	290 + 10	14° 46′	30
7	293	8° 58′	30
8	295 + 70	8° 8′	30
9	302 + 50	3° 20′	30
10	304 + 50	4° 49′	30
11	310 + 70	1° 0′	30
12	315 + 81.6	17° 15′	40
13	316 + 71.6	13° 39′	30
14	318 + 31.6	7° 25′	30

"The change of direction between successive sections of an elbow was limited to 4° or 5°, so that each section is about $2\frac{1}{2}$ or 3 ft. long. * * * The butt straps, sizes of rivets and rivet spacing are the same as for straight pipe."†

Throughout the series of observations on the steel pipe the upper pressure station was at the point marked *B* on the profile. The lower station, near the power house, was at *A* during observations 1–12, after which it was at *A′*. The length of pipe from *A* to *B* is 4 427 ft.; from *A′* to *B*, 4 367 ft. The diameter of the pipe varies slightly in different portions, the mean value being 72.22 ins.‡

Record of Observations.

(*1*) *Examples of Full Data Upon Which Tabulated Results Are Based.*—As already explained, the quantities recorded as constituting a single "observation" are in every case based upon readings taken at short intervals during a period of some length. These periods were selected after plotting, to a time base, the values of the four quantities ob-

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.
† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 262.
‡ *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 259.

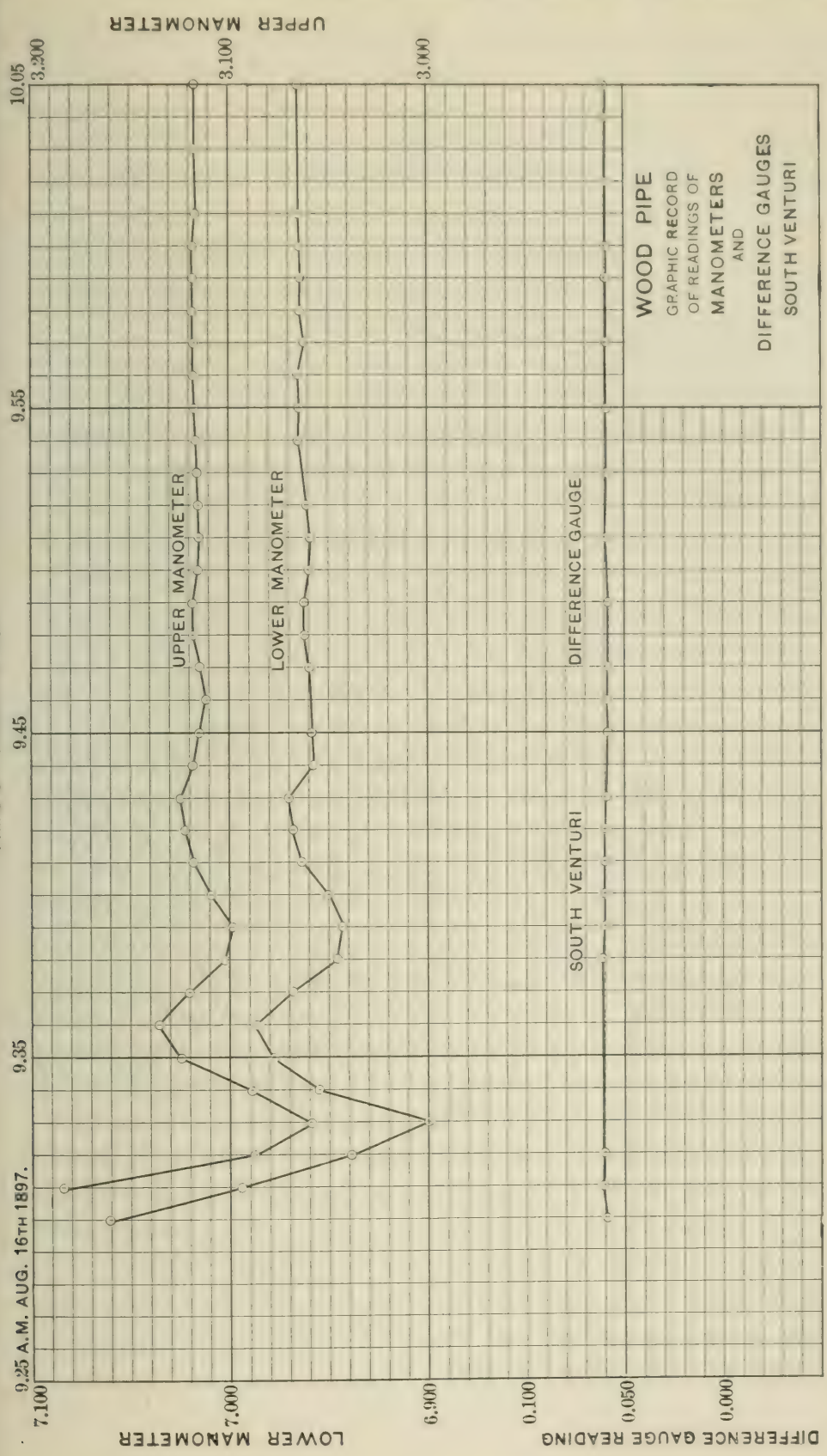

WOOD PIPE
GRAPHIC RECORD
OF READINGS OF
MANOMETERS
AND
DIFFERENCE GAUGES
SOUTH VENTURI
UPPER MANOMETER
LOWER MANOMETER
DIFFERENCE GAUGE
SOUTH VENTURI
TIME OF OBSERVATION
9.25 A.M. AUG. 16TH 1897.
9.35
9.45
9.55
10.05
UPPER MANOMETER
3.200
3.100
3.000
LOWER MANOMETER
7.100
7.000
6.900
DIFFERENCE GAUGE READING
0.100
0.050
0.000

FIG. 5.

served, namely, the heights of the two manometer columns (reduced for temperature), and the readings of the two difference-gauges attached to the Venturi meters (at times, however, only one meter was in use). The nature of the actual records obtained will be sufficiently indicated by showing samples of these plotted results. These are given in Figs. 4 and 5. Although the record given in Fig. 5 was taken during an experiment on the wooden pipe, it is representative of the character of the records obtained in both sets of observations, in all cases except the nine observations taken under conditions of falling pressure; while the record shown in Fig. 4 is representative of these nine observations. In these cases of falling pressure, periods were selected during which the difference-gauges showed the rate of discharge to be nearly uniform, while the manometer readings showed a nearly constant rate of decrease of the pressure at each pressure station. This rate of decrease at any given instant had practically the same value at the two stations, and the difference between the heights of the manometer columns thus remained constant during the time of an observation. In case of observations made under conditions of decreasing pressure, any discrepancy between the times of taking manometer readings at the two stations introduces an error into the observed value of the difference between heights of manometer columns. The instants of reading were as nearly coincident as they could be made by comparison of watches, and it is believed that the results are not affected by serious error. Assuming, as above, that the time of transmission of a pressure-wave between the two stations is less than one second, the oscillations of pressure furnish a check on the coincidence of the times of observation.

(2) *Tabulation of Reduced Observations.*—The entire series of experiments made upon the steel pipe resulted in twenty-nine "observations" whose periods were selected in the manner above described. The dates of these observations are shown in Tables Nos. 1 and 2.

The date and period of each observation are shown in columns 2 and 3 of Table No. 1. Columns 4 to 9 of the same table give the results of the observations on the Venturi meters. The "reading of difference-gauge," given in columns 4 and 6 for the two meters, is the difference between the observed heights of the two mercury columns connected respectively with the throat and the up-stream section of the Venturi. The equivalent water column ("head on Venturi") is

TABLE No. 1.—Steel Pipe Discharge Observations.

1	2	3	4	5	6	7	8	9
			North Venturi Meter.		South Venturi Meter.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Rate of discharge of main pipe (cubic feet per second).	Mean velocity of flow in main pipe (feet per second).
	1897.							
1....	Aug. 4	2.35— 2.45	.....	.....	0.022	14.5	14.5	0.510
2....	" 4	3.23— 3.31	.....	.....	.199	44.8	44.8	1.575
3....	" 6	2.10— 3.00	0.022	14.5	.....	.....	14.5	0.510
4....	" 6	3.15— 3.40	.093	30.9	.....	.....	30.9	1.086
5....	" 6	3.50— 4.15	.090	30.3	.....	.....	30.3	1.065
6....	" 7	1.33— 1.56	.....	.....	.225	47.5	47.5	1.670
7....	" 7	1.57— 2.15	.....	.....	.225	47.5	47.5	1.670
8....	" 7	2.45— 2.53	.....	.....	.512	71.2	71.2	2.503
9....	" 7	3.15— 3.29	.002	4.6*	.481	69.2	73.8*	2.594*
10....	" 7	4.11— 4.23	.181	42.6	.391	62.5	105.3	3.701
11....	" 7	5.00— 5.13	.078	28.2	.141	37.6	65.8	2.313
12....	" 7	5.15— 5.50	.077	28.1	.140	37.5	65.6	2.306
13....	" 9	1.42— 2.15	.127	35.8	.296	54.5	90.3	3.174
14....	" 9	2.36— 3.01	.090	30.3	.226	47.5	77.8	2.735
15....	" 9	3.26— 4.00	.333	57.8	.221	47.1	104.9	3.687
16....	" 9	4.12— 4.30	.323	57.0	.219	46.9	103.9	3.652
17....	" 11	8.40— 8.51	.021	14.1	.....	.....	14.1	0.496
18....	" 11	8.55— 9.15	.021	14.1	.....	.....	14.1	0.496
19....	" 11	9.30—10.00	.099	31.7	.....	.....	31.7	1.114
20....	" 11	10.39—10.43	.095	31.2	.043	21.0	52.2	1.835
21....	" 11	11.10—11.31	.086	29.6	.159	40.0	69.6	2.447
22....	" 11	11.35—12.00	.087	29.8	.160	40.1	69.9	2.457
23....	" 12	9.15— 9.28	.204	45.5	.....	.....	45.5	1.599
24....	" 12	9.29— 9.59	.205	45.7	.....	.....	45.7	1.606
25....	" 13	8.30— 8.50	.....	.....	.026	16.0	16.0	0.562
26....	" 13	9.13— 9.40	.111	33.7	.072	27.2	60.9	2.141
27....	" 13	10.07—10.40	.270	51.8	.123	35.3	87.1	3.062
28....	" 13	11.36—11.46	.401	63.2	.211	46.2	109.4	3.846
29....	" 13	11.48—12.03	.400	63.1	.210	46.1	109.2	3.839

* The rate of the discharge of the N. meter was so small during this observation that it could not be accurately measured.

found by multiplying the tabulated number by 12.6 or $e - 1$, e being the specific gravity of mercury. The values of the head on Venturi are not given in Table No. 1, but will be found in Tables Nos. 8 and 9. Columns 5, 7, 8 and 9 need no explanation. In computing values of the mean velocity in the main pipe, the diameter has been taken as 72.22 ins.

The manometer observations, with the values of the loss of head deduced from them, are recorded in Table No. 2. The headings of the columns explain sufficiently the meanings of the quantities tabulated. Column 2 gives the length of the main pipe between the points of at-

TABLE No. 2.—STEEL PIPE.—MANOMETER OBSERVATIONS.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	Heights of Manometer Columns Reduced to 0° Cent. (Feet.)			Static difference of mercury columns reduced to 0° cent. (Feet.)	Loss of Head between Manometers.		Loss of head per 1 000 ft. (in feet of water).
		Upper.	Lower.	Difference.		Mercury (Feet.)	Water. (Feet.)	
1..	4 427	11.251	33.916	22.665	22.668	0.003	0.041	0.0092
2..	4 427	11.059	33.686	22.627	22.668	.041	.558	.126
3..	4 427	11.258	33.914	22.656	22.668	.012	.163	.0369
4..	4 427	11.201	33.847	22.646	22.668	.022	.299	.0676
5..	4 427	11.204	33.847	22.643	22.668	.025	.340	.0768
6..	4 427	11.018	33.661	22.643	22.698	.055	.748	.169
7..	4 427	11.013	33.659	22.646	22.698	.052	.707	.160
8..	4 427	10.590	33.171	22.581	22.698	.117	1.591	.359
9..	4 427	10.559	33.136	22.577	22.698	.121	1.646	.372
10..	4 427	Dropping	Dropping	22.453	22.698	.245	3.332	.753
11..	4 427	10.720	33.300	22.580	22.698	.118	1.605	.363
12..	4 427	10.733	33.316	22.583	22.698	.115	1.564	.353
13..	4 367	10.296	32.816	22.520	22.707	.187	2.543	.582
14..	4 367	10.559	33.113	22.554	22.707	.153	2.081	.477
15..	4 367	Dropping	Dropping	22.472	22.707	.235	3.196	.732
16..	4 367	"	"	22.472	22.707	.235	3.196	.732
17..	4 367	11.260	33.960	22.700	22.707	.007	.095	.0218
18..	4 367	11.262	33.966	22.704	22.707	.003	.041	.0093
19..	4 367	11.207	33.896	22.689	22.707	.018	.245	.0560
20..	4 367	11.022	33.669	22.647	22.707	.060	.816	.187
21..	4 367	10.754	33.352	22.598	22.707	.109	1.482	.340
22..	4 367	10.748	33.342	22.594	22.707	.113	1.537	.352
23..	4 367	11.080	33.752	22.672	22.707	.035	.476	.109
24..	4 367	11.086	33.750	22.664	22.707	.043	.585	.134
25..	4 367	11.261	33.963	22.702	22.707	.005	.068	.0156
26..	4 367	10.890	33.520	22.630	22.707	.077	1.047	.240
27..	4 367	10.375	32.923	22.548	22.707	.159	2.162	.495
28..	4 367	Dropping	Dropping	22.458	22.707	.249	3.386	.776
29..	4 367	"	"	22.460	22.707	.247	3.359	.769

tachment of the two manometers. The point of attachment of the upper manometer remained unchanged during the experiments; that of the lower was changed after observation No. 12.* Column 6 gives the difference between the heights of the two manometer columns under static conditions. The observed value of this difference changed because the quantity of mercury in the lower manometer changed, for reasons to be considered presently. The numbers in column 7 are obtained from those in columns 5 and 6 by subtraction. The values in column 7 are reduced to equivalent water heights (column 8) by multiplying by 13.6; and the loss of head per thousand feet (column 9) is found from total loss of head (column 8) and length of pipe (column 2).

* The reason for this change was a suggestion that the point of attachment was at first too near the fork in the main pipe. The results, however, show no evidence of vitiation from this cause.

TABLE No. 3.—Steel Pipe. General Results.

1	2	3	4	5
No.	Mean velocity in main pipe. Feet per second.	Loss of head per 1 000 feet. (Feet.)	Value of c in formula $v = c\sqrt{rs}$.	Value of f in formula $H' = f \cdot \frac{l}{d} \cdot \frac{v^2}{2g}$.
17	.496	0.0218	87	0.0343
18	.496	.0093	132	.0147
1	.510	.0092	137	.0137
3	.510	.0369	68	.0549
25	.562	.0156	116	.0191
5	1.065	.0768	99	.0262
4	1.086	.0676	108	.0222
19	1.114	.0560	121	.0174
2	1.575	.126	114	.0197
23	1.599	.109	125	.0165
24	1.606	.134	113	.0201
6	1.670	.169	105	.0235
7	1.670	.160	108	.0222
20	1.835	.187	109	.0215
26	2.141	.240	113	.0202
12	2.306	.353	100	.0257
11	2.313	.363	99	.0262
21	2.447	.340	108	.0219
22	2.457	.352	107	.0226
8	2.503	.359	108	.0222
9	2.594	.372	110	.0214
14	2.735	.477	102	.0246
27	3.062	.495	112	.0204
13	3.174	.582	107	.0224
16	3.652	.732	110	.0212
15	3.687	.732	111	.0208
10	3.701	.753	110	.0213
29	3.839	.769	113	.0202
28	3.846	.776	113	.0203

(*3*) *General Results.*—In Table No. 3 are shown three sets of results, arranged for convenient comparison with other recorded experiments and accepted results. The observations are arranged according to values of v, the mean velocity of flow (column 2), and in columns 3, 4 and 5 are given the values of the three quantities most commonly used in estimating the carrying capacity of pipes. The quantities given in this table are also represented graphically in Figs. 6, 7 and 8.

Fig. 6 shows the loss of head per thousand feet as a function of the mean velocity of flow. The curve expressing this relation would be a parabola with vertex at the origin, and principal axis parallel to the ordinates representing loss of head, if c and f were constants.

In Fig. 7 each observation is represented by a point whose abscissa is v, and whose ordinate is the value of c in the Chezy formula, $v = c\sqrt{rs}$. In like manner Fig. 8 shows, for each observation, the value of v and that of f, the coefficient in the formula for loss of head,

$$H' = f \frac{l}{d} \frac{v^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

From an inspection of Figs. 6, 7 and 8 it is evident that the observations do not clearly disclose any law of variation of the coefficients c and f with the velocity of flow. The curves which appear to best represent the distribution of the plotted points are $A\ B$ (Fig. 7) and $A'B'$ (Fig. 8). [It is to be noticed that the value of f from observation No. 3 (0.0549) is so great as to throw the point beyond the limit of Fig. 8.] These curves show, between $v = 1.5$, and $v = 2.5$, a slight decrease in c and increase in f; and from $v = 2.5$ to $v = 4.0$, a slight increase in c and decrease in f. Such a law of variation cannot, however, be regarded as well established by the results shown.

It is apparent that, for values of v less than 4 ft. per second, the values of c and f do not greatly vary with the velocity. The average of all the values of c given in Table No. 3 is 109.1; omitting cases in which v is less than 1 ft. per second, the average is 109.4. The corresponding values of f are 0.0216 and 0.0215.

In addition to the above quantities, it may be well to record the values of n, the coefficient of roughness in Kutter's formula. The above mean value of c gives for n values ranging from about 0.013 to 0.015, the smaller values corresponding to low velocities of flow and the larger to the higher velocities.

Degree of Reliability of Results.

The reliability of the measurement of rate of discharge has already been discussed.

In the determination of loss of head in the steel pipe a difficulty was met which somewhat impaired the reliability of the results. This was the difficulty of making the manometer reservoir and the pipe containing the mercury column perfectly tight under the high pressure existing at the lower manometer station. After the experimenting had continued for several days, it was found that there had been a leakage of mercury sufficient to appreciably affect the manometer readings. As the fluctuations of the surface of the mercury in the reservoir could not be accurately measured, the changes in the quantity of mercury in the manometer introduced errors into the observed values of y (the difference between heights of mercury columns). It has already been pointed out that a constant error in these values is of no consequence; but the leakage of mercury introduced a variable error of uncertain amount. Had the difficulty been foreseen, means

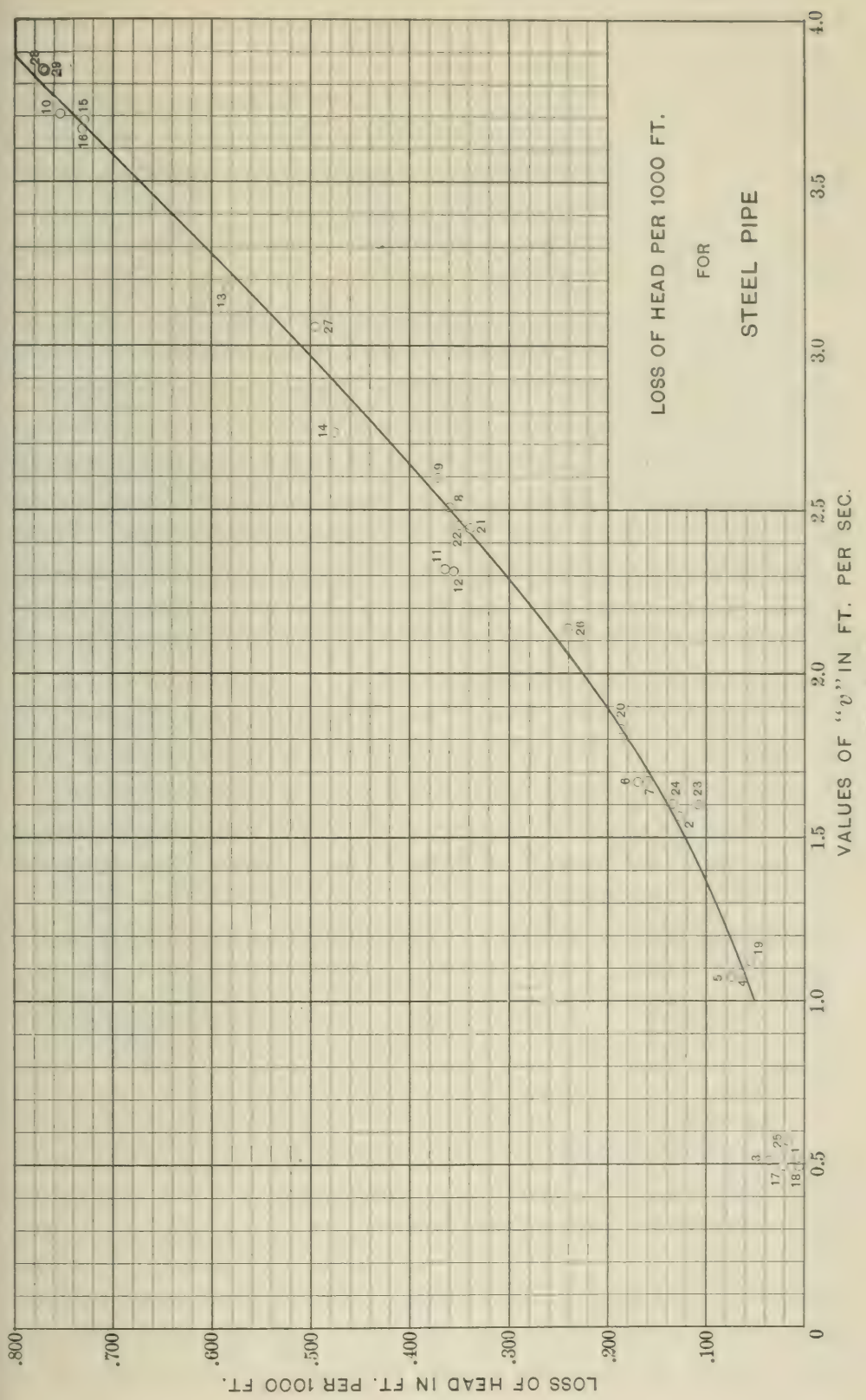

LOSS OF HEAD PER 1000 FT.
FOR
STEEL PIPE
LOSS OF HEAD IN FT. PER 1000 FT.
.800
.700
.600
.500
.400
.300
.200
.100
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 6.

would have been provided for reading accurately the position of the mercury surface in the reservoir.

A careful study of the results made it evident that the leakage was very slow, the only case in which a marked change in the static readings was noticed being after the manometer had remained under pressure over night. During the latter portion of the series of experiments, the leakage apparently decreased so much as to be of little importance.

Had it been possible to secure reliable readings of the heights of the manometer columns, under static conditions on each day on which observations were made, the effect of leakage could have been estimated and the errors due to it eliminated. This was not always possible, but a sufficient number of static readings was obtained so that a fair estimate could be made of the values of "static difference of mercury columns" to be used with the different observations. These values, as nearly as could be estimated, are given in column 6 of Table No. 2. The value of the "static difference" shows an increase at observation No. 6, and again at No. 13. This was due to the addition of mercury to the lower manometer.

It is believed that the greater part of the irregularity observed in the values of loss of head per thousand feet, and in the values of c and f, is due to the leakage of mercury from the lower manometer. A further possible cause of error, especially in the earlier observations, may have been the presence of air in the pipes leading from the main pipe to the manometers. In the case of the lower manometer, the connecting pipe was necessarily of considerable length, since the only practicable location for the manometer was in the power-house, and careful manipulation was needed to keep so long a pipe wholly free from air. This was probably accomplished more successfully during the later observations than during the earlier ones.

The possible errors due to changes of temperature of the mercury columns, and the method of correcting for such changes, have already been discussed. As a further possible source of error, changes of temperature of the water in the main pipe may be mentioned. It was not practicable to observe the temperature of the water within the pipe during the experimenting. The best that could be done was to take the temperature of the water in the waste flume below the power-house. No complete series of temperature observations was

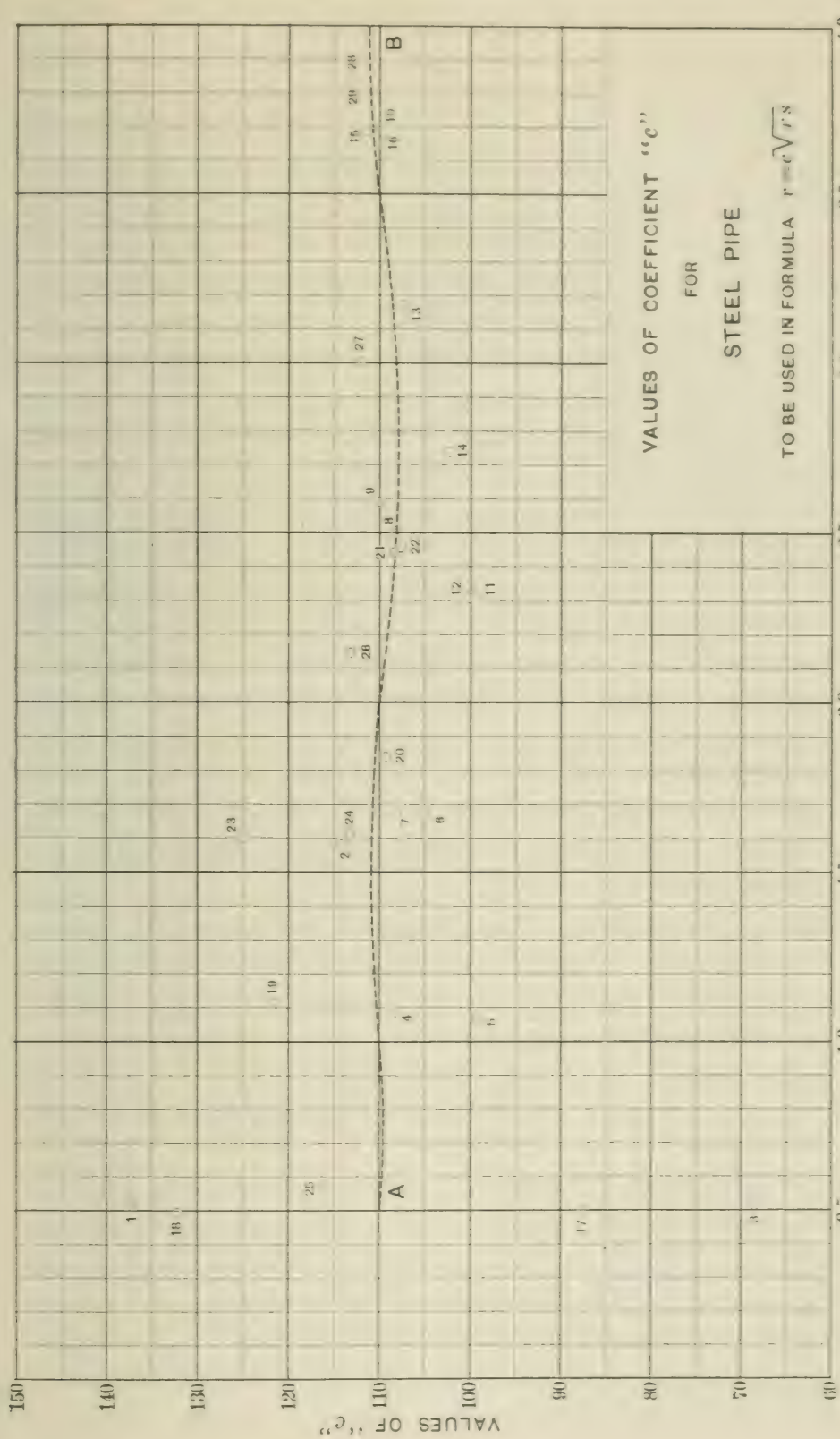

VALUES OF COEFFICIENT "c"
FOR
STEEL PIPE
TO BE USED IN FORMULA $v = c\sqrt{rs}$
A
B
VALUES OF "c"
VALUES OF "v" IN FT. PER SEC.
150
140
130
120
110
100
90
80
70
60
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

Fig. 7.

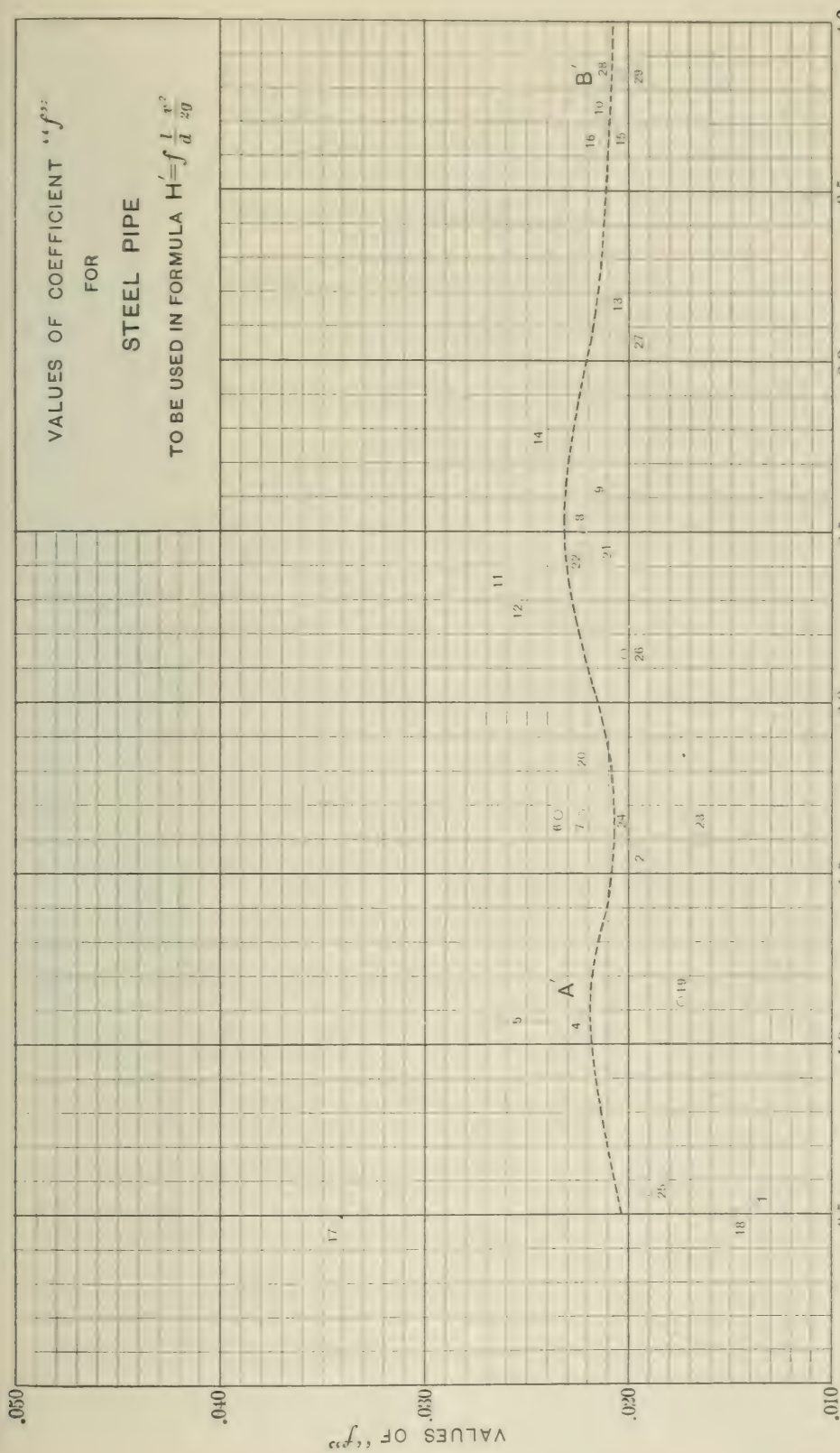
taken, but frequent observations, taken during the time from August 4th to August 11th, showed a variation of about 2° Cent., the range being from 16 to 18 degrees. The coefficient of expansion of water at this temperature is about 0.000174, nearly as great as that of mercury. A variation of the temperature of the water from 16° to 18° Cent. would change the value of the difference of pressure at the two manometer stations by about 0.1 ft. of water, and would appreciably affect the manometer readings, though its importance would not be great, except for small values of the loss of head measured.

Although the plotted results (Figs. 6, 7 and 8) indicate the presence of errors sufficiently large to be of importance in comparison with the quantities measured, yet it will be seen that the absolute values of the errors affecting the pressure measurements are not great. It was thought that the method adopted for measuring loss of head should give results reliable to within 0.1 ft. in absolute value, and it is believed that, even with the difficulties mentioned, the results actually reached possess a degree of reliability as high as the standard set. Thus, the greatest discrepancy in the whole series is shown by observations Nos. 1 and 3. The mean velocity has the same value in the two cases, while No. 1 gives the least and No. 3 the greatest of all the values found for the coefficient c. The values of the total loss of head differ by only 0.122 ft. (column 8, Table No. 2), and the actual error in each value is doubtless within the 0.1 ft. mentioned as the standard of accuracy expected.

It may be noted further, that, if the value of c is regarded as constant, and its value obtained by averaging all the values found, excluding cases in which v is less than 1 ft. per second, the probable error of this mean value is about 0.9.

Comparison of Known Data Regarding the Discharge of Riveted Pipes.

So far as known to the authors, no previous experiments upon the discharge of new riveted pipes of diameter as great as 6 ft. have been made. Experiments upon riveted pipes of smaller sizes have been too few to furnish a basis for establishing the law of variation of the coefficient c with the velocity, diameter and other conditions. The resistance to flow offered by the pipe doubtless depends upon various elements—thickness of plates, length of sections, frequency of rivets and size of



VALUES OF "v" IN FT. PER SEC.

FIG. 8.

TABLE No. 4.—Experiments on Riveted Pipes,

No.	1	2	3	4	5	6	7	8	9	10	11	12
Age.	5 yrs.	New.	5 yrs.	5 yrs.	New.	1 yr.	14 yrs.	14 yrs.	5 yrs.	New.	New.	14 yrs.
Diam. (ins.)...	11	11.25	13	15	16	17	24	24	26	33	35	36
Vel.	Coefficient *c*.											
0.5												
1.0												
1.3		101.3										
1.48												80.3
1.5												
2.0												
2.44												
2.5												
2.78		114.0										
3.0												
3.23												
3.27												
3.32							75.8	78.5				
3.5												
3.52											126.8	
3.87			121.6									
3.90												
3.96										123.2		
4.0												
4.38				111.6								
4.5												
4.58					112.3							
4.60			109.4									
4.71	107.1											
4.90		122.5										
4.93												
5.0												
5.5												
6.0												
6.09	110.6											
6.67		126.5										
6.84				117.8								
6.93	111.5											
6.96			113.4									
7.31				119.1								
8.46				119.0								
8.65			113.0									
8.66	113.4											
8.85		128.1										
10.02	115.5											
10.52		129.9										
10.59				121.6								
10.71			114.4									
12.09				121.3								
12.60									134.1			
20.14						131.1						
n	.010	.010	.011	.011	.011	.010	.017	.016	.010	.011	.011	.01

WITH DIAMETERS RANGING FROM 11 INS. TO 103 INS.

No.	13	13a	14	15	16	17	18	19	19a	19b	20	21	22
Age.	New.	4 yrs.	New.	New.	New.	New.	New.	New.	4 yrs.	4 yrs.	New.	New.	5 yrs.
Diam. (ins.)	36	36	38	38	42	42	42	48	48	48	48	72	103
Vel.	Coefficient c.												
0.5												110	126.5
1.0	86.0					96.0	101.0	101.2	78.0	97.2	97.1	110	116.6
1.3													
1.48													
1.5	90.6					103.0	102.8	105.4	84.6	100.8	98.7	111	112.7
2.0	95.2					107.9	104.3	108.8	89.6	103.3	100.3	110	110.3
2.44					115.9								
2.5	99.4					111.0	105.5	111.2	92.4	104.9	101.6	108	108.8
2.78													
3.0	103.3					112.6	106.4	112.8	93.0	105.3	102.2	108	107.7
3.23			114.0										
3.27			116.6										
3.32													
3.5	107.0					113.0	107.2	113.4	93.2	104.8	103.6	110	106.9
3.52													
3.87													
3.90				109.2									
3.96													
4.0	110.6					112.8	107.8	113.2	94.0	104.0	104.2	111	106.2
4.38													
4.5	114.0					111.8	108.2	112.4	94.2	103.7	104.7		105.6
4.58													
4.60													
4.71													
4.90													
4.93		106.3											
5.0	117.2					110.8	108.4	112.0	94.4	103.7	105.1		
5.5	120.4					110.2	108.5	111.7	94.7	103.7	105.2		
6.0	123.6					110.0	108.5	111.6	94.9	103.7	105.2		
6.09													
6.67													
6.84													
6.93													
6.96													
7.31													
8.46													
8.65													
8.66													
8.85													
10.02													
10.52													
10.59													
10.71													
12.09													
12.60													
20.14													
n	.014	.013	.013	.013	.013	.013	.013	.013	.016	.014	.014	.014	.015

rivet-heads, nature of interior coating, and the number, frequency and sharpness of bends or elbows. These various conditions differ so much in the pipes hitherto experimented upon that hardly any two sets of results are directly comparable. The available data, so far as known to the authors, for pipes not less than 11 ins. in diameter, are shown in Table No. 4. The values of c are arranged according to values of the diameter, and also of the mean velocity of flow. At the bottom of each vertical column is given the value of n, the Kutter coefficient of roughness, as computed from the corresponding series of experiments. These values have not been computed with great precision, being taken from Kutter's diagram; and in case of experiments covering a considerable range of velocities, the recorded value of n is intermediate between the extreme values given by the experiments. Additional particulars regarding the different series of experiments from which the tabular results are deduced will now be given.

1, 3 and 4. Experiments by Hamilton Smith, M. Am. Soc. C. E., at North Bloomfield, Cal., in October, 1876. Pipes of sheet iron, single riveted, made in lengths of about 20 ft., "stove-pipe" or taper joints. Coated by immersion in boiling asphaltum and coal tar. In use 5 years, but interior surfaces worn smooth. Length in every case about 700 ft. Discharge measured by weirs. Loss of head by leveling between surface of supply reservoir and outflowing stream, correcting for velocity head. Smith's "Hydraulics," p. 302.

2. Experiments by Darcy, 1849–51. New sheet-iron pipe, coated with bitumen; screw joints; length, 365.5 ft. Discharge determined by measurement in tanks; loss of head (probably) either by water piezometers or mercury gauges. The tabulated results, in English units, are taken from Smith's "Hydraulics," p. 226.

5. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Ore., Water-Works. Steel pipe, cylinder joints; 16 ins. is diameter of small sections; asphalt coating. Discharge measured by rise of reservoir surface in test of 18 hours; loss of head by open stand-pipes. The measurement of loss of head for steel pipe was indirect; the total loss in the combined lengths of 19 130 ft. of stave pipe and 16 416 ft. of steel pipe was measured, and at the same time the loss in 4 188 ft. of stave pipe alone. *Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 21.

6. Experiment by Hamilton Smith at Texas Creek, Cal. Pipe of sheet iron, made in lengths of 20 ft.; heavy coating of asphaltum and coal tar. Length, 4 439 ft. Discharge measured by weir, 5.5 ft. long; loss of head by leveling between surfaces of supply and discharge tanks. Smith's "Hydraulics," p. 311.

7. Experiment by George W. Rafter, M. Am. Soc. C. E., on old conduit of Rochester, N. Y., Water Works, July and August, 1890. Cylinder joints. Length of experimental section, 1 901 ft. Discharge determined by rise of reservoir surface; loss of head by water piezometers, *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 20.

The tabulated results are based upon a discharge of 6 742 000 galls. per day. Four gaugings of this conduit, made when it was new (in 1876), gave as an average 8 704 000 galls. per day.* Assuming equal ratio of decrease of capacity to have taken place in all parts of the conduit, the original gaugings would give about 97 as the value of c (for $v = 4.25$) when the pipe was new.

8. Same experiment as No. 7, but a different portion of the conduit. Length, 10 541 ft. Loss of head measured by Bourdon gauges.

9. Experiment by Hamilton Smith on "Humbug" pipe of North Bloomfield (Cal.) Mining Company. Pipe of sheet iron, coated with asphaltum and coal tar. Laid in 1868; experiment made in 1873, the surface being then quite smooth. Measurement of discharge not regarded as very exact. Loss of head measured by leveling between surfaces of supply and discharge reservoirs. Length, 1 194 ft. Smith's "Hydraulics," p. 309.

10. Experiment by Isaac W. Smith, M. Am. Soc. C. E., on conduit of Portland (Ore.) Water-Works. Cylinder joints; asphalt coating. Observations made on February 27th, 1896. Discharge (probably) measured by weir; loss of head by water piezometers. Length, 34 176 ft. *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 203.

11. Same experiment as No. 10; another portion of conduit. Length, 39 809 ft.

12. Same experiment as Nos. 7 and 8; another portion of conduit. Loss of head measured by water piezometers. Length, 50 819 ft.

The mean of the above-mentioned four measurements made in 1876 would give, for this section of pipe, a value of c of about 103 for $v = 1.9$; assuming equal ratio of decrease of capacity for all parts of the conduit.

13. Experiments by Clemens Herschel, M. Am. Soc. C. E., on conduit of the East Jersey Water Company, from Belleville to South Orange Avenue, 1892. Cylinder joints, asphalt coating. Discharge measured by "imperfect weir;" loss of head by Bourdon gauges. Length, 25 000 ft. Herschel, "115 Experiments on the Carrying Capacity of Large, Riveted, Metal Conduits," pp. 28, 52.

13*a*. Same conduit as No. 13; experiment by Herschel in 1896. Discharge measured by Venturi meter; loss of head by Bourdon gauges. Length, 24 720 ft. "115 Experiments," pp. 28, 52.

14. Experiments by Emil Kuichling, M. Am. Soc. C. E., on new conduit of Rochester Water-Works between Hemlock Lake and Rush

* "Annual Reports Executive Board, Rochester, N. Y." 1894–95, p. 169.

Reservoir, October and December, 1895. Cylinder joints, inner sections 38 ins. in diameter; part coated with asphalt, part with Sabin's japan. Length, 91 641 ft. Discharge measured by rise of reservoir surface; loss of head by reading mercury gauge at lower end and observing level of water in feeding chamber at upper end. "Annual Reports Executive Board, Rochester, N. Y.," 1894–95, p. 344.

A portion of this conduit (from West Bloomfield to Rush Reservoir, 34 468 ft.) was tested again in June and October, 1896, giving $c = 90.04$ as mean of 22 observations. Methods of measurement and range of velocities not stated; "total fall in hydraulic grade line" ranged from 1.08 ft. to 6.95 ft. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1896, p. 45.

15. Experiments by Emil Kuichling on new Rochester conduit between Rush Reservoir and Mt. Hope Reservoir, October and November, 1895. Construction same as No. 14; coated with Sabin's japan. The tabulated results are the mean of three gaugings agreeing closely among themselves. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1894–95, p. 349.

16. Same experiment as Nos. 10 and 11; another portion of conduit; construction the same. Length, 50 965 feet.

17. Experiments by Clemens Herschel on Kearney Extension of East Jersey Water Company. Taper joints, coating "unusually smooth." Conduit put in use January 10th, 1896; experiments made January 21st–30th, 1896 (except one November 18th, 1896). Length, 5 574 ft. Greatest velocity recorded, 4.26. Discharge measured by Venturi meter: loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

18. Experiments by Clemens Herschel on conduit No. 2 of East Jersey Water Company, below Pompton Notch. Conduit put in use September 30th, 1896; experiments made in September and October, 1896. Length from 49 833 to 81 139 ft. Highest recorded velocity, 5.41. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 29, 53.

19. Experiments by Clemens Herschel on Conduit No. 1 of East Jersey Water Company, 1892. Observations made on various portions of whole line, lengths ranging from 24 630 to 74 396 ft. Cylinder joints, asphalt coating. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 26, 27, 52.

19*a*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to section above Pompton Notch. Length ranging from 10 507 to 24 630 ft. Highest recorded velocity, 4.63. Discharge and loss of head measured as in No. 19. "115 Experiments," pp. 27, 52.

19*b*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to portion below Pompton Notch.

Length ranging from 26 610 to 83 000 ft. Highest velocity recorded, 6.06. "115 Experiments," pp. 27, 52.

20. Experiments by Clemens Herschel in 1896 on Conduit No. 2 of East Jersey Water Company, above Pompton Notch. Taper joints; length, 24 648 ft. Velocity ranged from 2.99 to 4.69. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

21. Experiments on conduit of the Pioneer Electric Power Company, Ogden, Utah, August, 1897. Butt joint, asphalt coating. Length, 4 367 ft. and 4 427 ft. Velocity ranged from 0.50 to 3.85. Discharge measured by Venturi meters; loss of head by mercury gauges.

22. Experiments by Clemens Herschel in 1887, on feeding trunk of Holyoke testing flume. Diameter nominally 9 ft.; actual diameter of mean section, 8.58 ft. Cylinder joints; paint coating worn off; rather rusty. Length, 152 88 ft. Discharge measured by "accurate weir"; loss of head by water piezometers (surfaces of water in still boxes determined by hook gauges). *Trans. Am. Soc. C. E.*, pp. 246, 247. "115 Experiments," pp. 29, 54.

It will be noticed that in the case of the authors' results (column 21, Table No. 4), the values of c are given only to the nearest unit, while in every other case the first decimal figure is given. It is not to be inferred from this that these results are regarded as less precise in character than the others. In the opinion of the authors no experiments have been made of such a character as to warrant the use of more than three significant figures in the values of c. In certain cases experimenters have recorded values of c to the sixth significant figure. Such an appearance of precision would be warranted if the results were reliable to within one-thousandth of 1 per cent.

Very little in the way of general conclusions as to the carrying capacity of large riveted conduits can be deduced from the data above recorded. It is evident that the capacity of such conduits is less than was formerly supposed. The experiments of Hamilton Smith on pipes of the smaller sizes (Nos. 1, 3, 4, 6 and 9, Table No. 4) had been supposed to warrant the conclusion that riveted pipes had nearly or quite as great carrying capacity as smooth cast-iron pipes. For such pipes it has been supposed that the value of c increases with the velocity and also with the diameter; the table and curves given by Hamilton Smith*

* "Hydraulics," p. 271 and Pl. XIV. From an inspection of Pl. XIV. it appears that the curves showing the relation between c and v for diameters greater than 1 ft. are based on very meager data, and that the curves as drawn do not conform to the few experimental points recorded. It seems to be arbitrarily assumed that the law of variation of c for large diameters must show a general agreement with the law determined by experiment for small diameters.

being regarded as safe guides in the selection of coefficients. The results recorded in Table No. 4 under numbers 13 to 22 indicate that for large riveted pipes Smith's values are much too great. It is also to be noticed that these results show little or no variation of c with the diameter, and no decided variation with the velocity. It would appear that, for conduits of 36 ins. diameter or over, it is unsafe to count on a value of c greater than 110 for new pipes; if there are frequent or sharp bends the coefficient will doubtless be still less. Moreover, in case of long conduits under ordinary conditions of use, it is to be expected that the capacity will suffer considerable diminution with age.

It should be said that at the time the Ogden conduit was designed, the views of the engineering profession as to the carrying capacity of large riveted conduits fully warranted the use of a coefficient of 120. The construction of the conduit was such that its "smoothness" would certainly appear to compare favorably with any previously made. The butt joints give an interior surface without breaks, there being no interior butt straps at the round joints, while the longitudinal straps are continuous throughout the length of the conduit. As regards smoothness, this construction would appear to have a distinct advantage over either "cylinder" or "taper" joints.

A remark may be made regarding the applicability of Kutter's formula. From Table No. 4 it appears that, for diameters less than 36 ins., and for pipe which is either new or in such condition as to be probably as smooth as when new, the value of n shows little variation in the different experiments, being 0.010 or 0.011 in every case. For larger sizes there is an increase of n with the diameter. This does not appear to be accounted for by a greater roughness of the larger pipes experimented on, but is rather to be attributed to the imperfection of the Kutter formula. It is further true that experiments on the same pipe with different velocities (and consequently different values of the slope s) give different values of n.

III.—Experiments on Wooden Pipe.

Description of Pipe Line.

The plan, profile, dimensions and construction of the entire pipe line may be found in the paper, already referred to.* The plan and

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

profile of the portion of the wooden pipe upon which the experiments were made are shown in Fig. 9, *C* being the lower and *D* the upper pressure station. The diameter of the pipe is 72.5 ins.

Record of Observations.

(*1*) *Example of Plotted Results.*—Reference has already been made to Fig. 5, showing graphically the record of the two manometers and of the Venturi meter difference-gauge for a part of the run made on August 16th on the wooden pipe. Four observations of the series made on the wooden pipe were taken at times of falling pressure, giving manometer records similar to that shown in Fig. 4, although the latter record belongs to an experiment on the steel pipe.

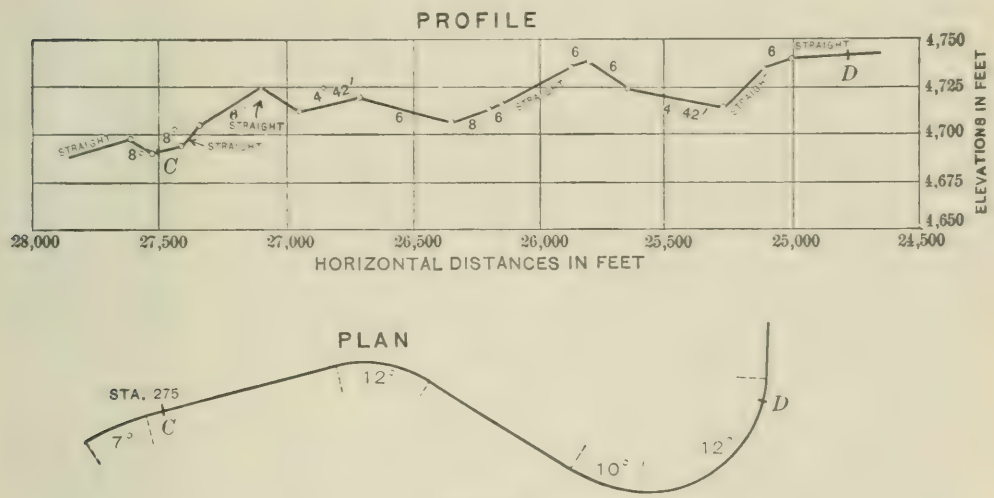


FIG. 9

(*2*) *Tabulation of Reduced Observations.*—From the experiments made upon the wooden pipe, 22 "observations" resulted. The reduced data of these observations are given in Tables Nos. 5, 6 and 7, of which no explanation is needed, since they are in all respects similar to the tables showing data of observations on the steel pipe (Tables Nos. 1, 2 and 3).

The date and period of each observation are given in Table No. 5. It will be noticed that observations 40, 44, 45 and 46 were made under conditions of falling pressure.

(*3*) *General Results.*—Table No. 7 shows the observations arranged in order of the values of the mean velocity of flow, three sets of results

TABLE No. 5.—Wooden Pipe. Discharge Observations.

1	2	3	4	5	6	7	8	9
			North Venturi Meter.		South Venturi Meter.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Rate of discharge of main pipe. (Cubic feet per second.)	Mean velocity of flow in main pipe. (Feet per second.)
	1897.							
30....	Aug. 15.	8.30— 9.00	.....	.....	0.054	23.4	23.4	.816
31....	" 15.	9.28— 9.45	.....	.....	.178	42.5	42.5	1.482
32....	" 15.	10.14—10.30	.....	.....	.376	61.3	61.3	2.138
33....	" 15.	10.50—11.15	.028	16.7	.366	60.4	77.1	2.689
34....	" 15.	11.32—12.00	.150	38.9	.363	60.1	99.0	3.453
35....	" 16.	8.01— 8.30	.....	.....	.024	15.3	15.3	.534
36....	" 16.	8.45— 8.52	.....	.....	.122	35.2	35.2	1.228
37....	" 16.	8.54— 9.15	.....	.....	.125	35.6	35.6	1.242
38....	" 16.	9.50—10.05	.....	.....	.406	63.6	63.6	2.218
39....	" 16.	10.26—10.45	.028	16.7	.400	63.1	79.8	2.783
40....	" 16.	11.00—11.30	.156	39.6	.395	62.7	102.3	3.568
41....	" 17.	8.30— 9.00	.....	.....	.036	19.1	19.1	.666
42....	" 17.	9.28— 9.45	.....	.....	.145	38.2	38.2	1.332
43....	" 17.	10.16—10.30	.....	.....	.321	56.7	56.7	1.978
44....	" 17.	10.48—11.15	.174	42.1	.390	62.4	104.5	3.645
45....	" 17	11.30—11.41	.174	42.1	.383	61.9	104.0	3.627
46....	" 17.	11.50—12.00	.174	42.1	.380	61.6	103.7	3.617
47....	" 18.	8.30— 9.00	.....	.....	.025	15.6	15.6	.544
48....	" 18.	9.15— 9.45	.....	.....	.120	34.9	34.9	1.217
49....	" 18.	10.25—10.30	.....	.....	.288	53.8	53.8	1.876
50....	" 18.	10.56—11.15	.032	18.1	.297	54.6	72.7	2.536
51....	" 18.	11.36—12.00	.130	36.2	.292	54.1	90.3	3.149

being given as in the case of the steel pipe already discussed. The quantities in this table (values of c, f and loss of head per 1 000 ft.) are also represented graphically in Figs. 10, 11 and 12.

These results differ from those found for the steel pipe in showing a well-marked variation of c and f with the velocity of flow.

Fig. 11 shows that c increases with the velocity in somewhat the way usually assumed for smooth cast-iron pipes. The highest velocity obtained in the experiments was about 3.64 ft. per second. The highest velocity, not accompanied by a fall of pressure, was about 3.45 ft. per second (observation 34). The four observations giving highest velocities are somewhat discrepant, due, probably, to the difficulty of determining accurately the difference between the heights of manometer columns during periods of decreasing pressure.

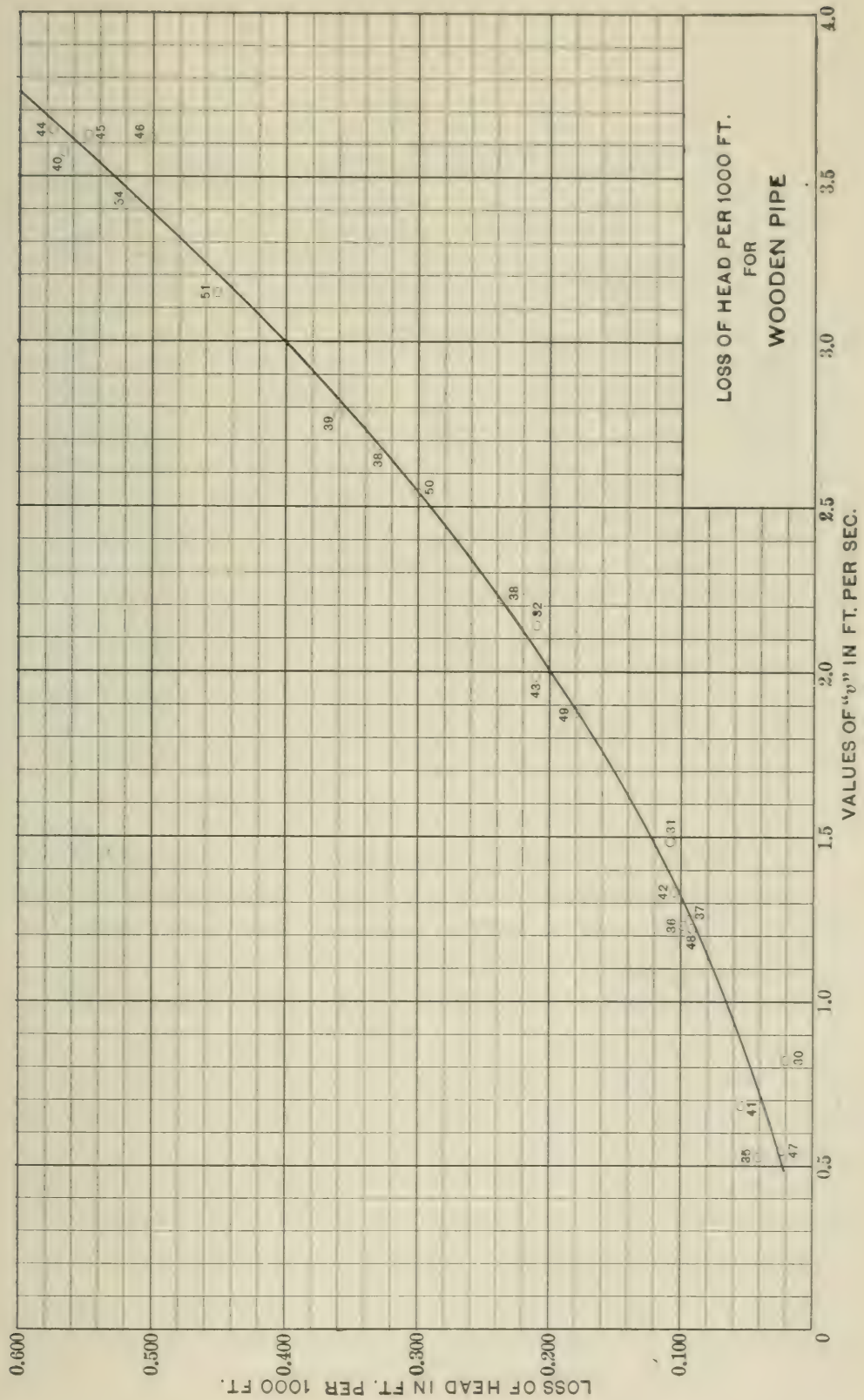

LOSS OF HEAD PER 1000 FT.
FOR
WOODEN PIPE
LOSS OF HEAD IN FT. PER 1000 FT.
VALUES OF "v" IN FT. PER SEC.
0.600
0.500
0.400
0.300
0.200
0.100
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
44
40
45
46
34
51
39
38
50
38
32
43
49
31
42
36
48
37
41
30
35
47

Fig. 10.

TABLE No. 6.—Wooden Pipe.—Manometer Observations.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	Heights of Manometer Columns Reduced to 0° Cent. (Feet.)			Static difference of mercury columns reduced to 0° Cent. (Feet.)	Loss of Head between Manometers.		Loss of head per 1 000 ft. in feet of water.
		Upper.	Lower.	Difference.		Mercury. (Feet.)	Water. (Feet.)	
30..	2 710	3.474	7.364	3.890	3.894	.004	.054	.0201
31..	2 710	3.384	7.256	3.872	3.894	.022	.299	.111
32..	2 710	3.151	7.003	3.852	3.894	.042	.571	.211
33..	2 710	2.882	6.710	3.828	3.884	.066	.898	.331
34..	2 710	2.434	6.225	3.791	3.894	.103	1.401	.517
35..	2 710	3.496	7.382	3.886	3.894	.008	.109	.0401
36..	2 710	3.446	7.320	3.874	3.894	.020	.272	.1003
37..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0952
38..	2 710	3.116	6.963	3.847	3.894	.047	.639	.235
39..	2 710	2.833	6.655	3.822	3.884	.072	.979	.361
40..	2 710	Dropping	Dropping	3.781	3.894	.113	1.537	.566
41..	2 710	3.486	7.369	3.883	3.894	.011	.150	.0552
42..	2 710	3.417	7.290	3.873	3.894	.021	.286	.106
43..	2 710	3.211	7.063	3.852	3.894	.042	.571	.211
44..	2 710	Dropping	Dropping	3.779	3.894	.115	1.564	.576
45..	2 710	"	"	3.785	3.894	.109	1.482	.547
46..	2 710	"	"	3.794	3.894	.100	1.360	.501
47..	2 710	3.492	7.381	3.889	3.894	.005	.068	.0251
48..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0953
49..	2 710	3.252	7.110	3.858	3.894	.036	.490	.181
50..	2 710	2.985	6.819	3.834	3.894	.060	.816	.300
51..	2 710	2.628	6.432	3.604	3.894	.090	1.224	.452
	....	3.518	7.412	3.894	3.894	.000	.000	.000

Reliability of Results.

Excluding observation 30, the experiments on the wooden pipe are more accordant than those on the steel pipe. This is, doubtless, due to the fact that the most important source of error in the steel pipe experiments was absent in the later series.

The greatest pressure to which either manometer was subjected in this series was due to a head of about 101 ft., as against 462 ft. in the former series. Under this lower pressure no difficulty was experienced in securing tight joints, and leakage of mercury did not occur. The long connecting pipe between the manometer and the main pipe (which was necessary at the lower end of the steel pipe) was also avoided, thus diminishing the difficulty of eliminating air. The temperature conditions were, on the whole, as favorable in the second series as in the first; for although neither manometer was furnished with a water jacket, the upper one was very completely sheltered from temperature changes, while the lower was not seriously exposed. The fact that

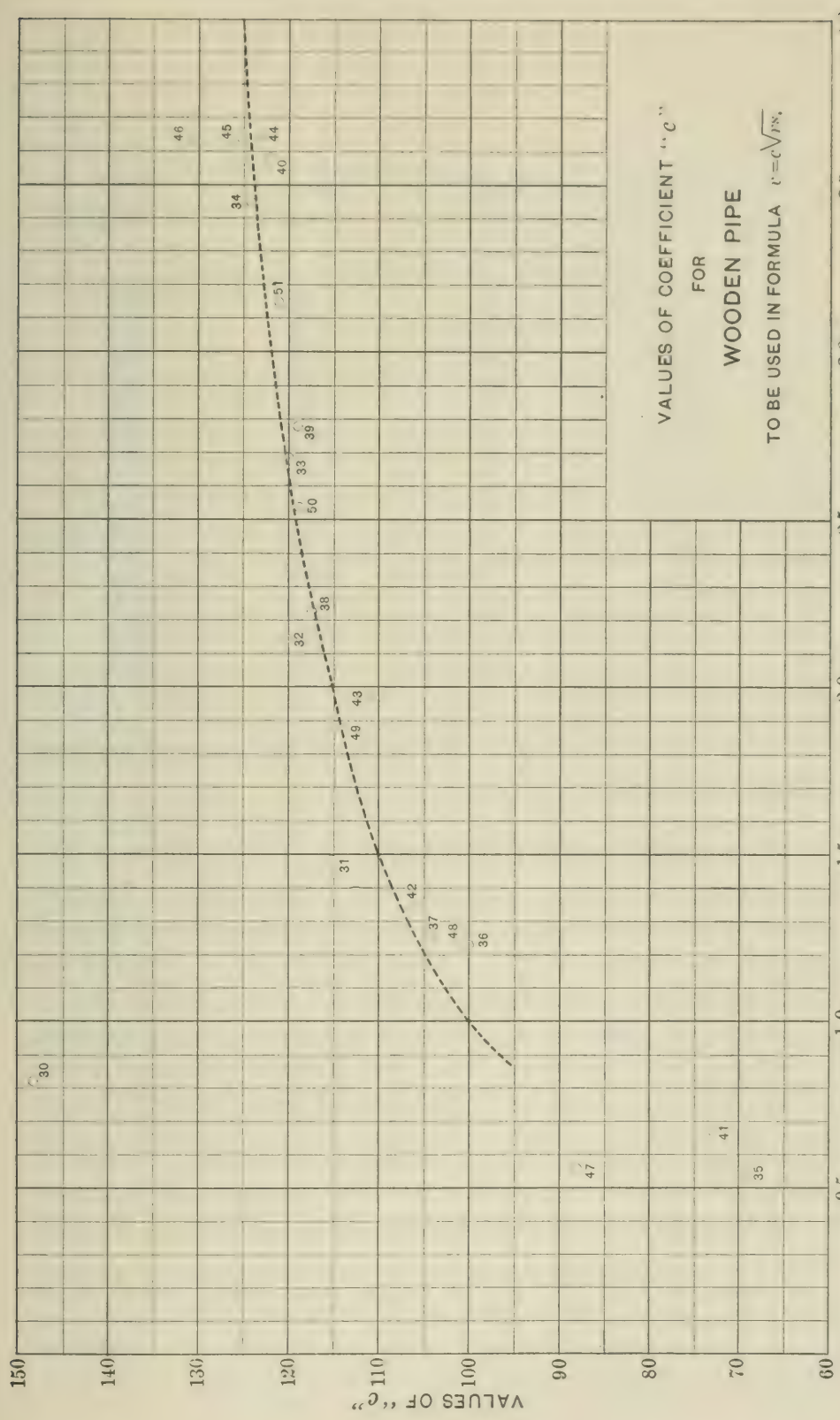

VALUES OF COEFFICIENT "c"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $v=c\sqrt{rs}$
VALUES OF "c"
150
140
130
120
110
100
90
80
70
60
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.
30
46
45
34
44
40
51
39
33
50
38
32
43
49
31
42
37
48
36
47
41
35

FIG. 11.

TABLE No. 7.—Wooden Pipe. General Results.

1	2	3	4	5
No.	Mean velocity in main pipe. (Feet per second.)	Loss of head per 1 000 ft. (Feet.)	Value of c in formula $v = c\sqrt{rs}$	Value of f in formula $H = f\frac{l}{d}\frac{v^2}{2g}$
35.....	.534	.0401	69	.0547
47.....	.544	.0251	88	.0262
41.....	.666	.0552	73	.0483
30.....	.816	.0201	148	.0117
48.....	1.217	.0953	101	.0250
36.....	1.228	.1008	100	.0259
37.....	1.242	.0952	104	.0241
42.....	1.332	.106	106	.0230
31.....	1.482	.111	115	.0195
49.....	1.876	.181	114	.0199
43.....	1.978	.211	111	.0209
32.....	2.138	.211	120	.0179
38.....	2.218	.235	117	.0186
50.....	2.536	.300	119	.0182
33.....	2.689	.331	120	.0178
39.....	2.783	.361	119	.0181
51.....	3.149	.452	121	.0177
34.....	3.453	.517	124	.0168
40.....	3.568	.566	122	.0173
46.....	3.617	.501	131	.0149
45.....	3.627	.547	126	.0161
44.....	3.645	.576	123	.0168

both columns were much shorter also rendered the temperature correction less important.

Known Data Regarding the Capacity of Wooden Pipes.

Besides the foregoing results, the authors know of only the following experimental data regarding the carrying capacity of wooden stave pipe.

1. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Oregon, Water-Works. This is the same experiment as that already described in connection with the discussion of steel pipes (No. 5, Table No. 4). The loss of head in 4 188 ft. of 18-in. stave pipe was measured by means of open stand-pipes. The discharge was determined by measuring the rise of the reservoir surface in a test of eighteen hours. The value of c was 132.9 for $v = 3.605$. The Kutter coefficient of roughness was about 0.010.—*Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 26.

2. In an account of the 30-in. stave pipe of the Denver Water-Works, by James D. Schuyler, M. Am. Soc. C. E., it is stated that gaugings were made by measuring the rise of the reservoir surface in a known time, and also by measurement of the velocity by current meter. As a result it is said that in "applying Kutter's formula to wood pipe, as

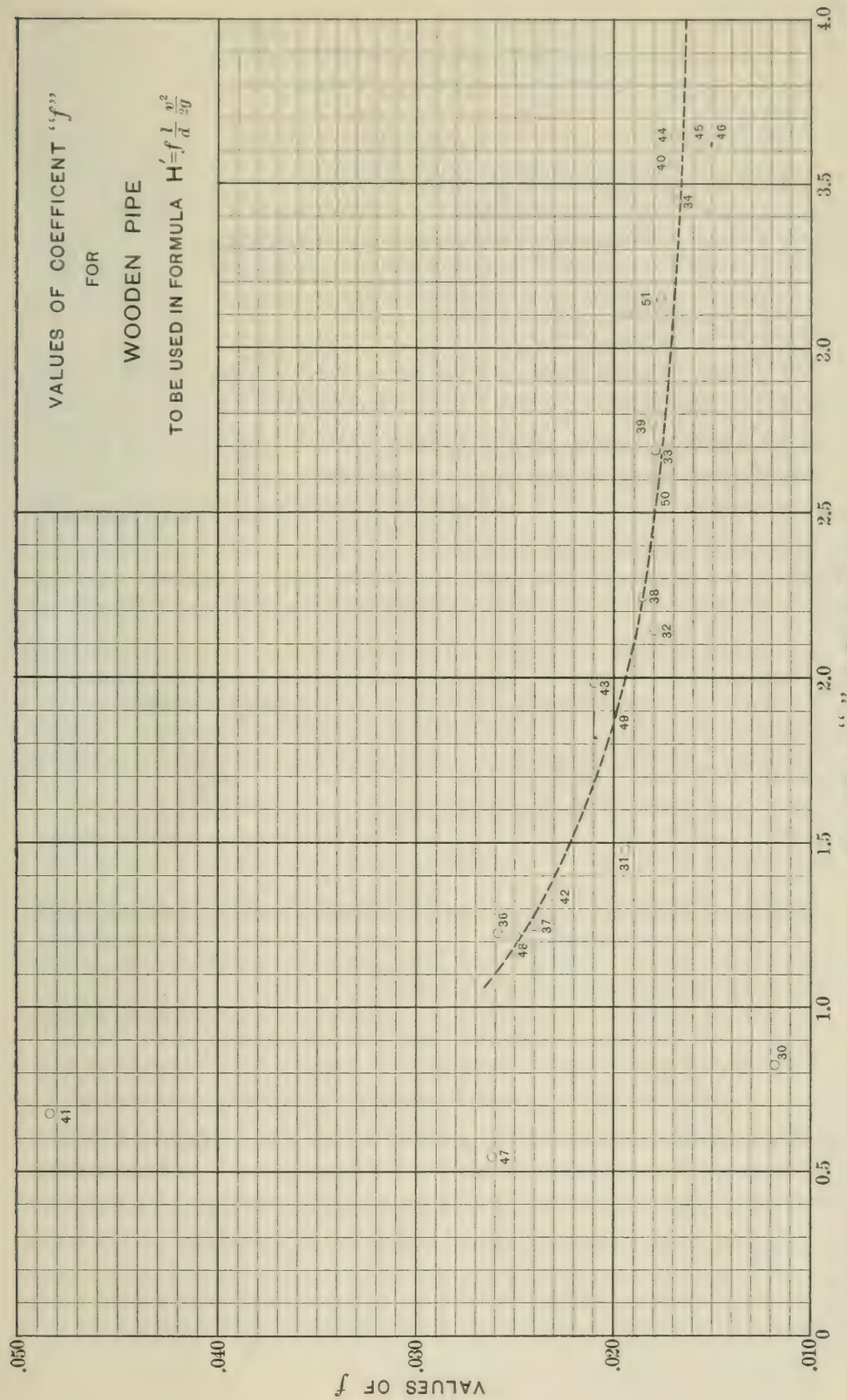

VALUES OF COEFFICENT "f"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $H'=f\frac{l}{d}\frac{v^2}{2g}$
VALUES OF f
.050
.040
.030
.020
.010
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
VALUES OF "v" IN FT. PER SEC.

FIG. 12.

low a coefficient n as 0.0096 can be used." This value of n would give values of c probably between 140 and 150, depending upon the hydraulic slope, which is not given. *Trans. Am. Soc. C. E.*, Vol. xxxi, p. 144.

3. Fred. B. Gutelius, C. E., states that in the design of the 24-in. stave pipe of the Butte City (Montana) Water Company, the value of n was taken as 0.010, and that the correctness of this value was justified by a test. The particulars of the test are not given, but this value of n would give values of c probably between 120 and 130, depending upon the hydraulic slope. *Journal of the Association of Engineering Societies*, Vol. xii, p. 219.

According to Hamilton Smith,* for circular pipe having "quite smooth interior surfaces, and no sharp bends," the value of c increases with the diameter and also with the velocity. Following are the values given by him for diameter 6 ft. and for velocities up to 4 ft. per second, with the corresponding values derived from the experiments on the Ogden pipe by the authors:

v	c	
	Smith's table.	Ogden experiments.
1	131.8	100
2	138.0	115
3	142.3	122
4	145.5	125

It will be seen that, while the two series agree in showing an increase of c with the velocity, the experimental values are decidedly lower than those given in Smith's table.

In this connection it may be noted that the experiments described by Desmond FitzGerald, M. Am. Soc. C. E.,† on clean cast-iron pipe 48 ins. in diameter, show a fair agreement with Smith's tables as regards the general law connecting v and c, but give values of c higher than Smith's for velocities less than 7 ft. per second. On the other hand, 48-in. pipe, with surface much tuberculated, showed a very different series of values of c; there being little variation of c with v except for low velocities, and the variation being a decrease of c with increasing velocity. The values of c for tuberculated pipe were not

* "Hydraulics," p. 271.

† *Transactions*, Am. Soc. C. E., vol. xxxv., p. 241.

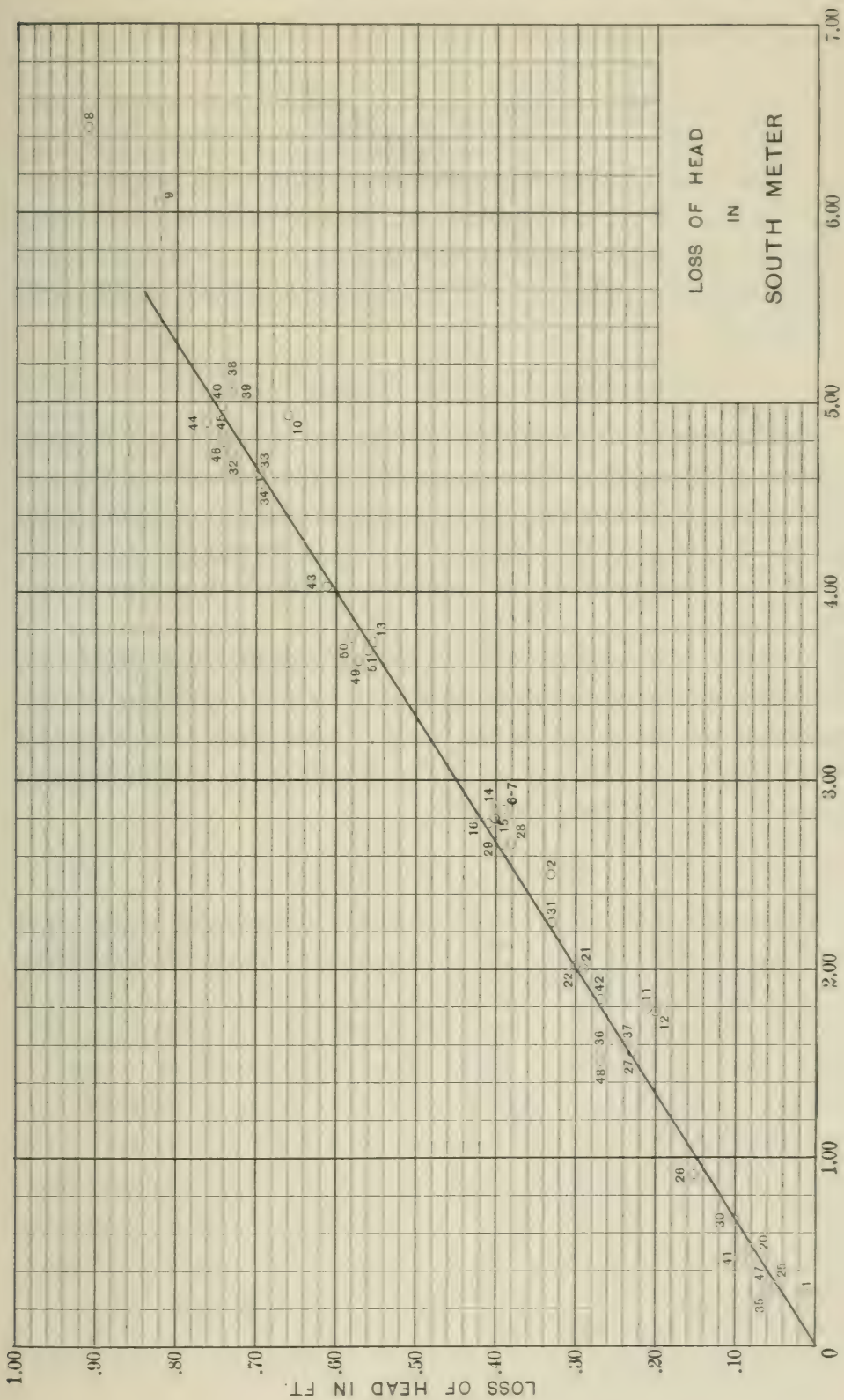

LOSS OF HEAD
IN
SOUTH METER
LOSS OF HEAD IN FT.
HEAD ON METER IN FT.
0
.10
.20
.30
.40
.50
.60
.70
.80
.90
1.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00

FIG. 13.

greatly different from the values for large riveted pipe given in Table No. 4.

In regard to the applicability of Kutter's formula it is to be said that the experiments on the wooden pipe herein described give values of n ranging from 0.012 to 0.015, an average value being perhaps 0.013. The difference between this value and those given for the Denver and Butte City conduits can hardly be attributed to the greater roughness of the Ogden pipe. It is rather to be supposed that the Kutter formula is defective.

IV.—Observations on Venturi Meters.

The Venturi meters being the only means available for measuring the rate of discharge of the pipe, no test could be made of the correctness of their indications, or for the determination of values of the coefficient to be used with them. As already stated, the authors assumed the correctness of the coefficients furnished by the manufacturers.

A matter of some importance, both theoretically and practically, is the loss of head caused by friction, and by the contraction and expansion of the stream within the meter. The values of this loss for different values of the rate of discharge were fairly well determined within the range of the experiments.

Loss of Head in Meters.

The results of the observations of loss of head in the meters have been tabulated for the two meters separately, Table No. 8 referring to the south meter, and Table No. 9 to the north meter. In each case the observations have been arranged in the order of increasing values of the rate of discharge. These tables show values of "head on Venturi," loss of head in meter, and rate of discharge of meter, for each observation. The values in column 2 are found by taking the difference between the readings of the mercury columns communicating with the throat and up-stream sections of the meter; while the values in column 4 are found in the same way from the up-stream and down-stream sections. The values in columns 3 and 5 are found from those in 2 and 4, respectively, by applying the factor 12.6, or $e - 1$, e being the specific gravity of mercury. The values in columns 2 and 6 are repeated from Tables Nos. 1 and 5.

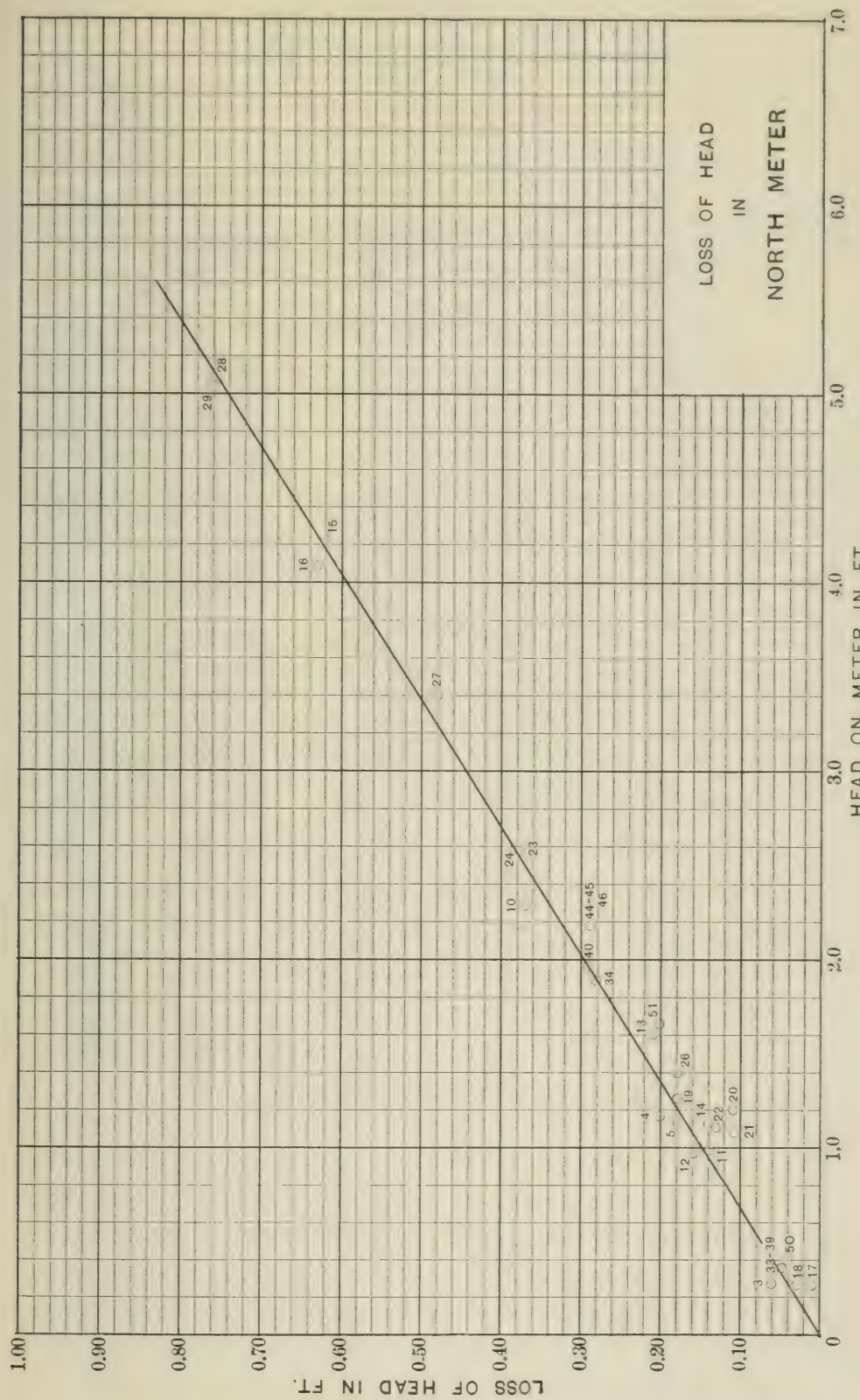

LOSS OF HEAD
IN
NORTH METER
LOSS OF HEAD IN FT.
HEAD ON METER IN FT.
0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.0
2.0
3.0
4.0
5.0
6.0
7.0

FIG. 14.

TABLE No. 8.—South Venturi Meter. Relation Between Rate of Discharge and Loss of Head in Meter.

1	2	3	4	5	6
Observation No.	Head on Venturi.		Loss of Head in Meter.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
1	.022	.28	.001	.01	14.5
35	.024	.30	.005	.06	15.3
47	.025	.32	.005	.06	15.6
25	.026	.33	.004	.05	16.0
41	.036	.46	.007	.09	19.1
20	.043	.54	.006	.08	21.0
30	.054	.68	.008	.10	23.4
26	.072	.91	.012	.15	27.2
48	.120	1.51	.021	.27	34.9
36	.122	1.54	.021	.27	35.2
27	.123	1.55	.018	.23	35.3
37	.125	1.58	.019	.24	35.6
12	.140	1.76	.016	.20	37.5
11	.141	1.78	.017	.22	37.6
42	.145	1.83	.021	.27	38.2
21	.159	2.00	.023	.29	40.0
22	.160	2.02	.024	.30	40.1
31	.178	2.24	.026	.33	42.5
2	.199	2.51	.026	.33	44.8
29	.210	2.65	.031	.39	46.1
28	.211	2.66	.030	.38	46.2
16	.219	2.76	.033	.41	46.9
15	.221	2.79	.032	.40	47.1
6	.225	2.84	.031	.39	47.5
7	.225	2.84	.031	.39	47.5
14	.226	2.85	.032	.40	47.5
49	.288	3.63	.045	.57	53.8
51	.292	3.68	.044	.56	54.1
13	.296	3.73	.042	.53	54.5
50	.297	3.74	.046	.58	54.6
43	.321	4.04	.048	.61	56.7
34	.363	4.57	.055	.69	60.1
33	.366	4.61	.055	.69	60.4
32	.376	4.74	.058	.73	61.3
46	.380	4.79	.059	.74	61.6
45	.383	4.83	.059	.74	61.9
44	.390	4.91	.060	.76	62.4
10	.391	4.93	.052	.66	62.5
40	.395	4.98	.059	.74	62.7
39	.400	5.04	.058	.73	63.1
38	.406	5.12	.058	.73	63.6
9	.481	6.06	.065	.82	69.2
8	.512	6.45	.072	.91	71.2

The simultaneous values of "head on Venturi" and "loss of head in meter" are plotted, for each meter separately, in Figs. 13 and 14. The irregularities shown in the diagrams are doubtless due chiefly to the presence of dirt at the top of the down-stream mercury column, as already explained. In spite of these irregularities the results show quite satisfactorily the relation between loss of head and head on Venturi. This relation, within the range of the observations, is well rep-

TABLE No. 9.—North Venturi Meter. Relation Between Rate of Discharge and Loss of Head in Meter.

1	2	3	4	5	6
Observation No.	Head on Venturi.		Loss of Head in Meter.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
17	.021	.26	.001	.01	14.1
18	.021	.26	.002	.03	14.1
3	.022	.28	.005	.06	14.5
33	.028	.35	.004	.05	16.7
39	.028	.35	.004	.05	16.7
50	.032	.40	.003	.04	18.1
12	.077	.97	.013	.16	28.1
11	.078	.98	.011	.14	28.2
21	.086	1.08	.009	.11	29.6
22	.087	1.10	.010	.13	29.8
14	.090	1.13	.011	.14	30.3
5	.090	1.13	.014	.18	30.3
4	.093	1.17	.016	.20	30.9
20	.095	1.20	.009	.11	31.2
19	.099	1.25	.014	.18	31.7
26	.111	1.40	.014	.18	33.7
13	.127	1.60	.017	.21	35.8
51	.130	1.64	.016	.20	36.2
34	.150	1.89	.022	.28	38.9
40	.156	1.97	.023	.29	39.6
44	.174	2.19	.023	.29	42.1
45	.174	2.19	.023	.29	42.1
46	.174	2.19	.023	.29	42.1
10	.181	2.28	.029	.37	42.8
23	.204	2.57	.030	.38	45.5
24	.205	2.58	.030	.38	45.7
27	.270	3.40	.038	.48	51.8
16	.323	4.07	.050	.63	57.0
15	.333	4.21	.049	.62	57.8
29	.400	5.04	.060	.76	63.1
28	.401	5.06	.060	.76	63.2

resented by a straight line. In other words, the loss of head in the meter appears to be directly proportional to head on Venturi. The loss of head thus appears to vary nearly as the square of the velocity of flow through the meter.

If H denotes head on Venturi and H'' loss of head in meter, the above relation is expressed by the equation $H'' = a\,H$, where a is a constant. The value $a = 0.149$ for each meter agrees well with the experimental data.

The relation between loss of head in meter and rate of discharge, for the Ogden meters, using the relation $H'' = 0.149H$, is shown by the curve in Fig. 15.

The experience of the authors in these tests has convinced them of the great value of the Venturi meter in experimental work, requiring

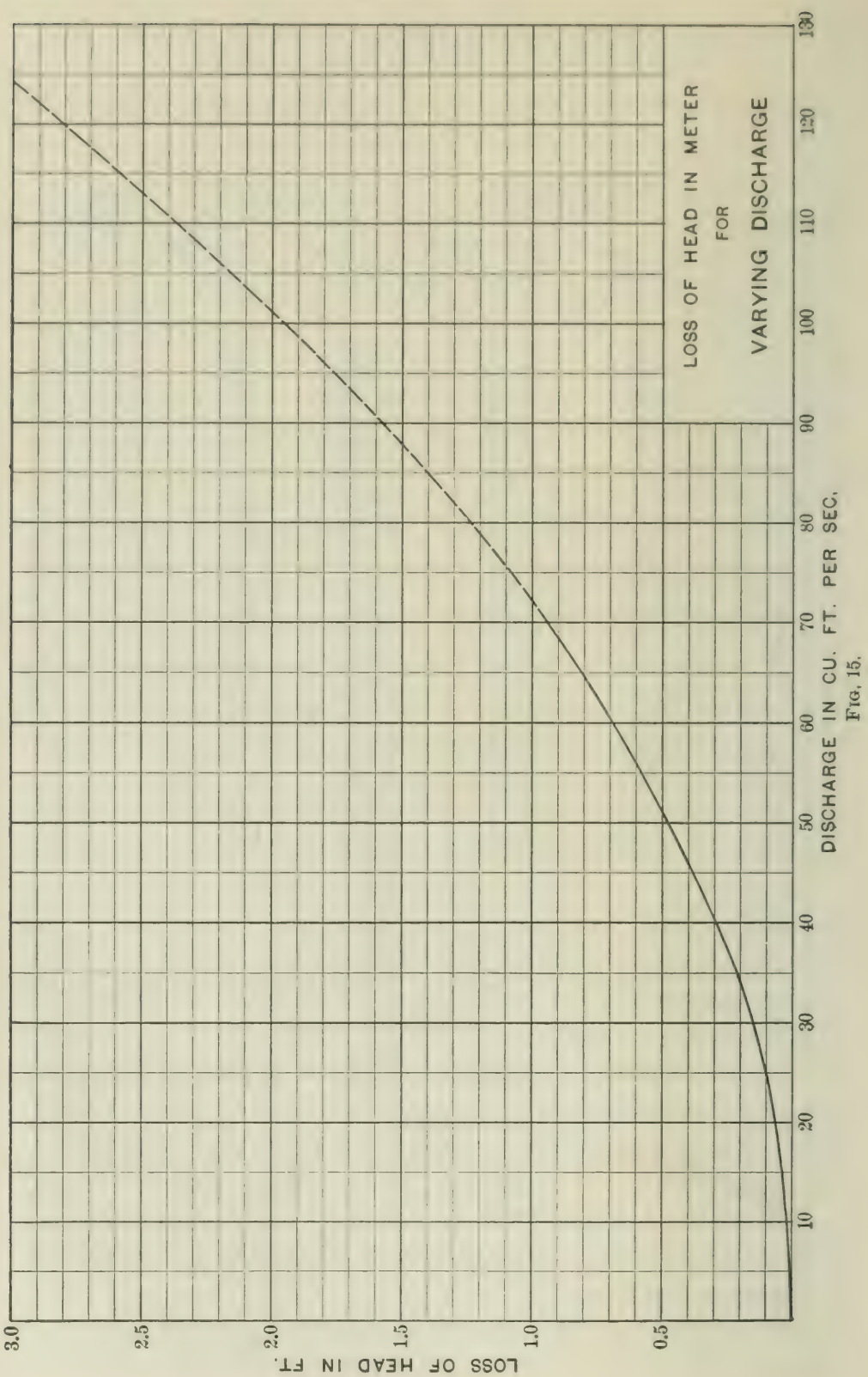

LOSS OF HEAD IN METER
FOR
VARYING DISCHARGE
LOSS OF HEAD IN FT.
3.0
2.5
2.0
1.5
1.0
0.5
DISCHARGE IN CU. FT. PER SEC.
10
20
30
40
50
60
70
80
90
100
110
120
130

FIG. 15.

the measurement of the rate of discharge of pipes. By no other method would it have been possible to collect so much information, at all comparable in accuracy with that obtained in these experiments, without months of labor. Moreover, the sensitiveness of the difference-gauge to slight changes of the rate of discharge shows that the meter is capable of giving measurements of great precision. It is perhaps not too much to say, that, except for very small quantities, the Venturi meter furnishes the most precise of all methods of measuring the rate of discharge, provided the values of the coefficient are accurately known.*

In carrying out these experiments the authors were assisted by Mr. L. S. Boggs, E. E., Superintendent of the Pioneer Electric Power Company; by Mr. F. W. Hart, Assistant Superintendent, and by Mr. L. B. Spencer, student in civil engineering in Stanford University. To them, for the valuable and efficient aid rendered; to the directors of the Pioneer Electric Power Company, and especially to Chief Engineer Bannister, for all courtesies rendered, the authors desire to express their sincere thanks.

* In regard to the range of the coefficients of a Venturi meter, the only experimental results known to the authors are those of Herschel. These results, obtained with meters of diameters of 12 ins., 48 ins. and 108 ins., indicate that the values of the coefficient, for throat velocities between 5 ft. and 25 ft. per second, are not likely to differ from unity by more than 3% (see "115 Experiments," Pl. III. p. 43). Even with an unrated meter it is probable that the results obtained will compare favorably in accuracy with those obtainable by any other method of measurement.

APPENDIX.

ATTACHMENT OF PIEZOMETERS.

Object of discussion.

It is proposed to consider the question whether the height of a piezometer column depends upon the position of the point of attachment to the pipe; and also whether different results will be given by multiple attachment than by attachment at a single point. The discussion will refer to a vertical cross-section of a horizontal pipe; but the reasoning will be seen to apply to the transverse section of a pipe having any direction.

Variation of Pressure in a Cross-Section.

The pressures at different points in the same cross-section are unequal, being less near the top of the pipe than near the bottom. If the particles of water all moved in straight lines parallel to the axis of the pipe, the law of variation of pressure with depth would be the same as for water at rest. Any variation from this law must be due to the fact that particles are deflected from the straight axial direction; in other words, to the fact that some of the particles of water passing a given section at any instant are being accelerated vertically. If such is the case, the distribution of the vertical components of acceleration throughout the cross-section will continually vary; so that not only will the law of variation of pressure with depth differ from the hydrostatic law, but the actual pressure at any given point will vary. Therefore, in any case in which a piezometer is found to give results which fluctuate but slightly, it may be concluded that the sinuosities in the motions of the particles of water have no important effect upon the law of variation of pressure in the cross-section. Experience bears out the supposition that this condition is satisfied when steady flow has been maintained for some time in a straight pipe of uniform cross-section. The following discussion will therefore proceed on the assumption that the pressure in any cross-section varies with the depth, according to the hydrostatic law.

Case of Open-Topped Piezometer.

If the pressure throughout a cross-section exceeds that of the atmosphere, and if open tubes are attached at two points, as at A and B (Fig. 16), it is evident that water will rise to the same level in both. For since the pressure at B exceeds that at A by an amount equiva-

lent to the height (h) of A above B, the corresponding pressure columns must differ in height by the same amount h. In fact, the pressure at any two points C and D at the same level in the two pipes must be equal.

If the pressure at the top of either column is less than atmospheric, that column will stand correspondingly higher ; but the surfaces X and Y will be at the same level so long as they are subject to equal pressures.

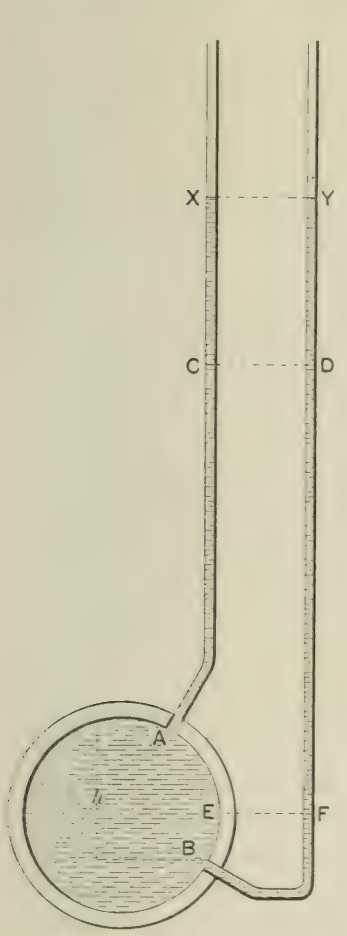


Fig. 16.

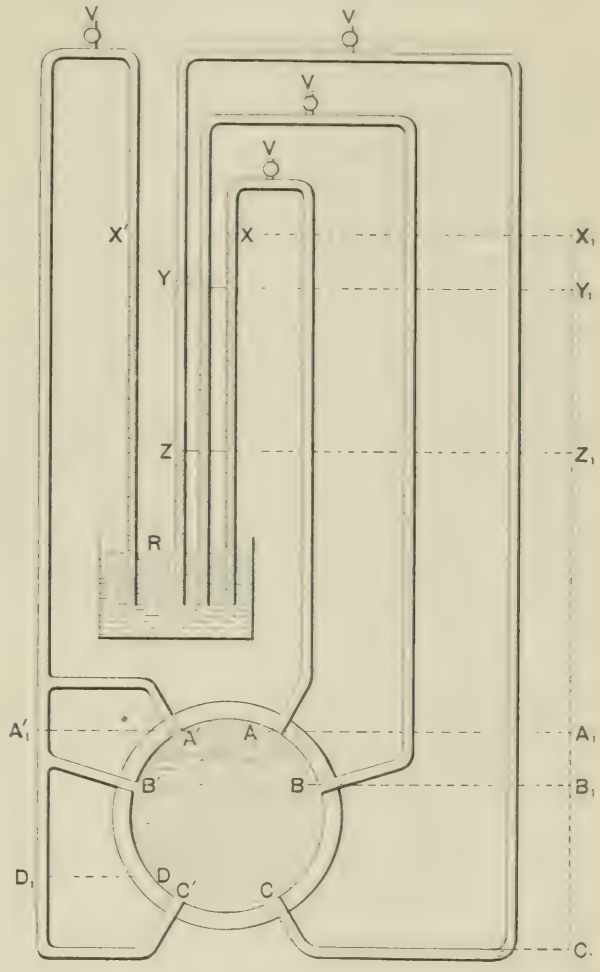


Fig. 17.

Case of Vacuum Gauge.

In case the pressure throughout the cross-section of the pipe becomes less than one atmosphere, the open piezometer becomes inapplicable. It is, however, instructive to consider what will happen if, with open tubes attached as in Fig. 16, the pressure within the pipe gradually diminishes. The surfaces X and Y will fall equally until the pressure within the pipe at the level A becomes just one atmosphere, the surface Y then standing on a level with A. The press-

ure at any point lower than A is still greater than one atmosphere. Let the pressure in the pipe diminish still further until there is just one atmosphere of pressure at some level E, between A and B; pressures less than one atmosphere existing at all points higher than E, and greater than one atmosphere at all points lower. The surface Y will fall to F, level with E; while at A air will be drawn into the pipe. If the level of atmospheric pressure falls below B, air will also enter at B.

Suppose, next, that the tubes attached at A and B are arranged as shown in Fig. 17, both tubes dipping into a reservoir whose surface is under atmospheric pressure, the pressure within the pipe being less than atmospheric in all parts of the cross-section. At first let air be admitted to the tubes through the valves V V, which are then closed. So long as air enters freely, the tubes act in the same way as the open tubes of Fig. 16. The pressure within the piezometer tubes being greater than that within the pipe, air enters the pipe from both tubes. After the valves V V are closed, the withdrawal of air causes a decrease of pressure in each tube, and water rises from the reservoir to a height which, for each tube, represents the difference between the pressure within the tube and that of the atmosphere at the surface of the reservoir. Air will continue to enter the pipe at A so long as the pressure within the tube exceeds the water pressure at A. A condition of equilibrium will finally be reached in which the pressure throughout the air in the tube is equal to the water pressure at A, and the surface X stands at a height above the reservoir surface representing the excess of atmospheric pressure above the pressure within the space A X. A like condition will be reached in the other tube, but since the pressure-head at B exceeds that at A by an amount A_1 $B_1 = h$, the surface Y will stand lower than X by an amount X_1 $Y_1 = h$.

It is further to be noticed that, if the opening at A is of appreciable size, water will stand in the tube up to the level of the top of the opening. Thus, suppose the exhaustion of the air within the tube has proceeded until the air pressure in the tube is greater than the water pressure at the top of the opening and less than that at the bottom; air will enter at the top, and water will flow out at the bottom, a condition of equilibrium being reached only when no part of the stream in the main pipe flowing past the orifice is in contact with air. Similarly, water will stand in the other tube up to the level of the top of the opening B.

It thus appears that it makes no difference at what point of the cross-section the piezometer is attached, provided the indicated pressure be referred to the proper level. If the apparatus is attached and operated in the manner above described, the piezometer reading in each case gives the pressure within the pipe at points which are on a level with the top of the orifice leading from the pipe to the piezometer tube.

If, after the condition of equilibrium above described has been reached, the pressure within the pipe increases, but still remains less than that of the atmosphere, water will rise in the tubes at A and at B, the surfaces X and Y at the same time falling. A condition of equilibrium will ensue in which the height of the column RX exceeds that of the column above A by an amount equivalent to the difference between atmospheric pressure and the pressure at A, with a similar condition of affairs in the tube attached at B. In such a case both columns in the same piezometer tube must be observed in order that the pressure at a given level in the main pipe may be known.

A further case to be considered is that in which some part of the piezometer tube is lower than the opening into the pipe, as at C, Fig. 17, the pressure within the pipe at C being supposed less than one atmosphere. If the tube is at first filled with air under atmospheric pressure, air will enter the pipe at C until the pressure within the tube falls to the value of the water pressure at the bottom of the opening. As soon as the air pressure becomes less than this value, water will flow into the tube through the lower part of the orifice, collecting in the lowest part of the tube until its cross-section is entirely filled. Water will continue to flow into the tube at the bottom of the orifice, and air to enter the pipe at the top, until no air remains in contact with the water flowing past the orifice in the main pipe. A condition of equilibrium will finally be reached in which water fills the tube from C to some point C_1. This point C_1 will be high enough so that the lowest cross-section of the tube is completely filled, but its exact height cannot be predicted. The height of the column $R\,Z'$ will measure the amount by which atmospheric pressure exceeds the pressure within the pipe at the level C_1.

Consider next the case of a single piezometer communicating with the given cross-section of the pipe at two points, as at A' and C', Fig. 17, the pressure at every point of the cross-section being less than that of the atmosphere. As before, let the valve V be first open and then closed. While it is open, air will enter the pipe at A' and at C'. After it is closed air will continue to enter the pipe so long as the pressure within the tube exceeds the water pressure, either at A' or at C'. At the same time water will rise from the reservoir R to a height measuring the amount by which the pressure within the tube is less than one atmosphere. When the pressure within the tube becomes less than that in the pipe in the lower portion of the opening C', air will cease to enter at this part of the opening, and water will flow into the tube. At the top of the opening air will still enter until the air pressure falls below the water pressure at that point; after which water will flow out through the entire opening. Air will continue to enter the pipe at A', and the air pressure in the tube to decrease. When the pressure in the tube is equal to the water pressure at some point $B\,D$, between A'

and C', water tends to stand in the tube at the height D_1; no condition of equilibrium is reached, however, so long as air continues to enter at A'. Finally the pressure in the tube will reach the value of the water pressure at the highest point of the orifice A', air will cease to enter the main pipe, and water will stand in both branches of the tube to the level $A'\ A'_1$. The air pressure in the tube is now equal to the pressure in the water at the level A', and is less than atmospheric pressure by an amount equivalent to the water column $R\ X'$.

Summary of Results.

The conclusions to be drawn from the foregoing discussion may be summarized as follows:

(1) When the pressure in the given cross-section of the pipe everywhere exceeds that of the atmosphere, an open piezometer will stand at the same height at whatever point of the cross-section it be attached, and whether it communicates with the pipe at one point or at several.

(2) If a "vacuum" piezometer be used, the pressure at the point of attachment being less than that of the atmosphere, the pressure at any level in the pipe can be inferred if the water surfaces in both branches of the piezometer are known. The pressure within the main pipe, at all points on a level with the surface of the column in the branch of the piezometer tube adjacent to the pipe, has "negative" value measured by the height of the column in the other branch above the open reservoir.

(3) In case of a vacuum piezometer connected with the pipe at more than one point in the same cross-section, water will stand at the same level in all the connecting tubes.

(4) In case of a vacuum piezometer, if air be freely admitted to the tube before the observation, the pressure within the main pipe remaining constant, water will rise in the connecting tubes, whether one or several, to the level of the highest point of communication with the pipe.*

The authors have not been able to subject these conclusions to the test of experiment. They are, however, in conformity with the results noticed and discussed by Herschel.† For measuring pressures in the throat of a Venturi meter he used a vacuum piezometer communicating with the throat at several points, the several branch tubes being furnished with valves, so that communication could be opened through any one or more of them at pleasure. From the working of the piezometer with different combinations of tubes open, Herschel concludes that:

"The Venturi must, in all cases, be pierced for connection with the air chamber vertically at its crown, and may be pierced radially at as

* Unless the tube leading to this point runs below the level of the opening into the pipe.

† *Transactions*, Am. Soc. C. E., Vol. xvii, pp. 248, 250, 251.

many additional points as we please, without affecting the reading of the standard crown orifice."

The foregoing theory suggests no reason for the conclusion that the pipe must be pierced at the highest point of the cross-section; but Herschel's conclusion is otherwise in strict conformity with the principles reached above, and summarized in the paragraphs numbered (3) and (4). These principles are not fully tested by the experiments recorded, since in every case bearing upon the point under discussion one of the open passages was the one at the top of the cross-section.

For the case of pressures greater than atmospheric, Herschel's experiments of October, 1887,* furnish several instances in conformity with the conclusion above drawn and stated in paragraph (1). This is, in fact, the conclusion reached by Herschel. In the experiments of the authors at Ogden no pressures less than atmospheric had to be measured.

The foregoing discussion has not referred to the effect of air carried by the water and collecting in the piezometer tube. Attachment of the piezometer at the top of the pipe probably gives greater liability to trouble from this source than attachment at a lower point; but in all cases means must be provided for getting rid of whatever air may collect.

* *Transactions*, Am. Soc. C. E., Vol. xvii, p. 250.

DISCUSSION.

Mr. Williams. GARDNER S. WILLIAMS, Assoc. M. Am. Soc. C. E.—Some information in regard to the calibration of the instruments used in the experiments under discussion, if they were calibrated at all, would seem to be desirable. The speaker has found that there is sometimes a variation in the specific gravity of mercury, and also that some instrumental coefficients may possibly enter into the question, notably in the connection with the lower gauge. The statement that an air-cock is provided at the highest point in this connecting pipe offers an opportunity for suspicion as to the results afforded by that gauge. From some observations which the speaker has made, he has learned that there is nothing more delusive than an air-cock. If it is not placed at the high point, or if the air bubble is not at the highest point and properly blown off, the entire test for that velocity and for that time may be vitiated, and, unless the observers were able to assure themselves that there was no accumulation of particles of air in the connection of their gauge, little dependence can be placed on the results they have given. It seems to the speaker that the lack of coherence of the observations and the apparent sinuosity of the mean line for the value of n, indicate that there was something very materially the matter with the observations. He does not wish to appear unduly critical of those who carried out these experiments, fully appreciating the difficulties under which they labored, yet he does not think these experiments are entitled to rank with the experiments heretofore made and recorded by Messrs. Desmond FitzGerald, John R. Freeman, and others. The same objection might be made to most of the experiments made in the West, on which dependence is placed for knowledge as to the carrying capacity of large conduits. They have been ordinarily single experiments by inexperienced experimenters, who are not aware of the conditions likely to be met, and the results obtained are not entitled to the same credence as the observations of those who have devoted years of study to the subject. On the line under discussion the effect of curvature must have been of some importance. Without wishing to go into that subject very extensively, the speaker desires to state positively that the line of maximum velocity of the water passing around a curve is not in the axis of the pipe. He fully agrees that more extended experiments are highly desirable, and would also suggest that, with shorter lengths and more delicate instruments, possibly more satisfactory results could be obtained.

Mr. Trautwine. JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—Mr. Herschel, in his paper* describing the Venturi meter as he designed it in the first

* *Transactions*, Am. Soc. C. E., Vol. xvii, p. 228, and Vol. xviii, p. 133.

place, seems to assume that the elevation of the columns in the piezometers might properly be measured from the top of the pipe; but Professor Merriman* remarks: "It is thought that the elevation should be measured to the center of the section rather than to the top." Prior to the appearance of the present paper, these expressions seem to have comprised all the published matter on the subject. The authors have, therefore, rendered valuable service in entering into an elaborate discussion of this point in the appendix to their paper. Mr. Trautwine

In Philadelphia, where the Venturi meter, in sizes up to 48 ins., has been extensively used, experiments have been made comparing the loss of head through different meters, the Venturi being compared with several of the ordinary mechanical meters. In these experiments the Venturi gave, at a given velocity, much less resistance to flow than any of the others, with the single (and somewhat singular) exception of the "Gem" meter, which, at high velocities, gave even a shade less resistance than the Venturi.

In all, the resistance increased in proportion to the head, or to the square of the velocity.

The range of velocities within which any given Venturi meter will work satisfactorily is rather limited, and it is important, therefore, to determine in advance, to what range of velocities the main in question is to be subjected, and to design the throat ratio of the Venturi accordingly, a high throat ratio (throat small relatively to main) being necessary to secure sufficient difference between the piezometer heights under low velocities, and a low throat ratio (throat wide relatively to pipe) being required to avoid throttling or "wire drawing" under high velocities.

Hence, when the Venturi is placed in a main where the normal velocities are low, and where the main may yet be occasionally drawn upon for abnormally high velocities, as in case of fire, a by-pass should be provided in order that the Venturi may be cut out of service at such times of abnormal demand.

HENRY GOLDMARK, M. Am. Soc. C. E.—In the Venturi meter, as used at Ogden, there are four piezometer openings; that is, a complete chamber surrounds the Venturi tube, and this chamber is tapped for the pressure pipe, but communicates with the interior of the pipe, at the top, the bottom and the two sides. Mr. Goldmark.

With reference to Mr. Henny's discussion, the speaker is familiar with some matters which, perhaps, deserve mention. One is with reference to the amount of sand that could have been brought into the pipe. The pipe was not really put into use until May, 1897, and it is very doubtful whether there was a continuous flow through it until some time in June. The experiments were made in August, so that the period of time during which there was a chance for sediment

* "A Treatise on Hydraulics," 1889, p. 161.

Mr. Goldmark. to enter was very short. It is true that the blow-off valves would probably not remove such sediment. It is also true that the so-called air-valves were not intended primarily for removing the air which might accumulate at summits. Their construction is such that the air cannot escape automatically, though it might be possible to release it by pressing on the top of the valves with a bar. It would not be easy to do so, and there is nothing in the paper to show that it was done before the experiments were made. The main purpose of these valves was to act as a safeguard in case of a break in the pipe, as they are designed to admit air at a high velocity to take the place of the water flowing off, and thus prevent the collapse of the pipe.

Mr. Kuichling. E. KUICHLING, M. Am. Soc. C. E.—This paper is an extremely valuable addition to hydraulic literature, and the experiments described therein can fairly be ranked with the best that have yet been made with large pipes. The authors appear to have done their work with close attention to the many details that are essential to accuracy, and have left little room for criticism. Their only fault, in the speaker's opinion, consists in not having explained sufficiently the reason for having adopted a uniform value of 13.60 for the specific gravity of the mercury which they used in their manometers. By this omission they may either entrap a reader of the paper into expressing dissatisfaction with this value, or involve him in a tedious examination of the subject, inasmuch as the authors' method of computing the equivalent height of a column of water from the observed height of a column of mercury appears at first sight to be somewhat inaccurate.

It is stated in the paper that the temperature of the water in the pipe ranged from 16° to 18° Cent., and that the observed heights of the mercury columns in the several manometers, at various other temperatures not given, were reduced to equivalent heights at the temperature 0° Cent.; also that these latter heights were then multiplied uniformly by the specific gravity (13.60) of the mercury, in order to obtain the height of the equivalent water columns at the aforesaid temperatures. Now, to those who are aware that the specific gravities of both mercury and water vary with considerable changes in the temperature of these liquids, this manner of obtaining the equivalent height of the water column cannot fail to appear to be of doubtful accuracy. On further investigation, however, such doubt will disappear, particularly when it is remembered that the manometers or mercury columns are not used to measure great differences in elevation, but only the comparatively small losses of head by friction in the large pipe; and it may also be remarked incidentally that the ingenious manner in which the authors have eliminated the constant difference in level between the manometer stations, and left in view only the friction loss of head in the pipe, is worthy of unstinted praise.

When an open mercury column is used to determine, from the

water pressure in a pipe, the difference in elevation of the corresponding points of attachment, it becomes necessary to pay close heed to the respective temperatures of the water and mercury, and perhaps also to calibrate the manometer under the particular conditions of temperature prevailing at the time of the observations. As already mentioned, the relative specific gravity of mercury varies with the temperature, and to exhibit this variation within the range of temperatures usually encountered in water-gauging operations, Table No. 10 is submitted. The weights per cubic foot of distilled water at the different temperatures, given in column 3, are taken from the carefully prepared paper on the subject by Mr. A. F. Nagle, M.E., read before the American Society of Mechanical Engineers in May, 1892, and the weight of pure mercury at 0° Cent., has been taken at 848.75 lbs. per cubic foot, as given by all well-recognized authorities: Mr. Kuichling.

TABLE No. 10.—Exhibiting Relative Specific Gravities and Weights per Cubic Foot of Mercury at Different Temperatures; also Weights per Cubic Foot of Distilled Water at Different Temperatures.

1	2	3	4	5	6	7	8	9
Temperature.		Weight of 1 cu. ft. of water. Lbs.	Weight of 1 cu. ft. of Mercury, by Regnault's formula. Lbs.	Relative specific gravity of Mercury.	Weight of 1 cu. ft. of Mercury, as per approximate rule of authors. Lbs.	Relative specific gravity of Mercury.	Weight of 1 cu. ft. of Mercury at 0° Cent. Lbs.	Relative specific gravity of Mercury.
Cent.	Fahr.							
0°	32°	62.4162	848.750	13.5982	848.750	13.5982	848.75	13.5982
4°	39°.2	62.4250	848.143	13.5866	848.139	13.5865	"	13.5936
5°	41°	62.4229	847.991	13.5846	847.986	13.5845	"	13.5968
10°	50°	62.4074	847.231	13.5758	847.222	13.5757	"	13.6002
15°	59°	62.3745	846.472	13.5708	846.458	13.5706	"	13.6073
20°	68°	62.3238	845.714	13.5697	845.695	13.5694	"	13.6184
25°	77°	62.2553	844.955	13.5724	844.931	13.5720	"	13.6334
30°	86°	62.1688	844.198	13.5791	844.167	13.5786	"	13.6523
35°	95°	62.0668	843.440	13.5892	843.403	13.5886	"	13.6748

Furthermore, to obtain the weight of 1 cu. ft. of mercury at other temperatures, as given in column 4, use was made of Regnault's formula for the cubical expansion of mercury,

$$v = v_0 (1 + 0.000179007\, t + 0.0000000252\, t^2),$$

where v_0 and v are the volumes at 0° and t° Cent., respectively; and to compare the results thus obtained with those derived from the simple approximate formula used by the authors, viz.: $v = \frac{v_0}{1 - 0.00018t}$, columns 6 and 7 have been computed. Column 9 gives the specific gravity of mercury at 0° Cent., relatively to water at the various other temperatures.

Mr. Kuichling. An examination of the last column shows that when the temperature of the water in the pipe is between 0° and 15° Cent., no appreciable error will be made by using the constant factor 13.60 (or the relative specific gravity of mercury at 0° Cent.) in order to convert a maximum height of only a few inches of mercury at 0° Cent. into the equivalent height of a column of water of somewhat higher temperature, as was done by the authors. If, however, the manometers were used to determine the difference in level between the two stations with the water at 20° Cent., for example, the use of the factor 13.60 instead of 13.6184 as per Table No. 10, would cause, with a mercury column 22.707 ft. high, an error of 0.418 ft. in the equivalent height of the water column, which would be intolerable in so short a length of pipe as 4 367 ft. It therefore follows that for exact leveling operations with open mercury manometers, either the correct value of the relative specific gravity of the mercury must be used, or the instruments must be carefully calibrated under conditions of temperature similar to those which prevailed during the observations. In general, for exact work, such a calibration is preferable to the use of the aforesaid theoretical factors or specific gravities, since account is thereby taken directly of any impurities in both the water and the mercury.

An instance of the use of an open mercury manometer to ascertain the difference in level between two points nearly 10.3 miles apart on a water conduit may perhaps be of interest in this connection. The experiment was carried out under the writer's direction in October, 1897, for the purpose of securing a check upon three sets of levels, which had been run at different times over the route of the conduit as correctly as possible with good wye-levels, and which did not agree with each other as closely as was deemed desirable for hydraulic observations. The difference in elevation of these two points was about 57 ft., and the elevations of the lower point given by the levels were + 248.370, + 249.230 and + 248.550, the average of these three elevations being + 248.717, and the difference between the first and second being 0.86 ft. The manometer was accordingly set up at the lower station several hours before readings were commenced, so that the mercury had ample time to acquire the same temperature as the air, and it was also placed so as to remain shaded; but as the day was cloudy, this precaution was not necessary. A good aneroid barometer was likewise provided at each station, in order to note whether any appreciable differences in the atmospheric pressure occurred. No such difference, however, was found. Observations were begun about half an hour after the outlet valve of the conduit was closed, and were continued at frequent intervals during a period of 46 minutes, in which the mean temperature of the air (and presumably also the mercury) was 7° Cent., while that of the water was 15° Cent., and the height of the mercury column was 4.170 ft. At these temperatures

the relative specific gravity of mercury is 13.5903, so that the corresponding height of the water column was 56.6716 ft.; and by using this value along with the other data pertaining to the case, the true elevation of the lower point becomes +248.788. On the other hand, if the simpler approximate method of reduction adopted by the authors had been followed in this case, the computed height of the water column would have been 56.6406 ft., thus making the required elevation 0.031 ft. less. Mr. Kuichling

It may also be remarked that the foregoing manometric leveling operation was subsequently checked by a direct calibration of the gauge, with the mercury, water and air all at a practically uniform temperature of 7° Cent., the instrument with its attached thermometer being packed in cotton waste which was kept saturated with a small stream of water from a faucet. The relative specific gravity of the mercury thus found agreed, up to the fifth decimal place, with that which was computed in the manner set forth in Table No. 10, and hence the terminal elevation mentioned has been accepted as correct, the inference being that both the water and the mercury were practically pure and followed exactly the indicated laws of variation in weight per unit of volume with changes in temperature. It should also be added that the manometric work above described was supplemented, on the remaining 9.4 miles of the conduit, by the direct observation and measurement of the static head of water as its surface stood exposed in a piezometer vessel located on the hillside at the lower end of the section; but as the further details of this case are not pertinent to the subject under discussion, they are accordingly omitted.

In their summary of available data relating to riveted pipes, the authors reach the conclusion that "for conduits of 36 ins. diameter or over, it is unsafe to count on a value of c greater than 110 for new pipes"; and, "if there are frequent or sharp bends the coefficient will doubtless be still less." Without desiring to criticise this general statement, the speaker ventures to express the belief that by taking proper account of the various resistances to the flow of the water which are peculiar to riveted pipes, the values of the coefficient c will become much less discordant than they now appear. No good reason can be given why there should be more friction on a smooth asphaltic coating of a steel plate than on the similar pitch coating of a cast-iron pipe, or on the smooth and bare surface of a well-made wooden pipe; but when it is considered that the riveted pipe presents on its interior a vast number of projecting rivet heads and plate edges, all of which represent variations in the diameter of the conduit and consequent small losses of head; also that changes in direction must necessarily be made by a series of abrupt deflections instead of by continuous curves of large radius, it is not surprising to find a marked difference

Mr. Kuichling. in the value of c in favor of cast-iron or wooden pipes. If, however, these numerous small resistances or losses of head in a riveted pipe are properly estimated and deducted from the observed total head or fall, so as to place such a conduit on a comparable basis with a similar line of cast-iron or wooden pipe, it will doubtless be found that the differences in the resulting values of c will be quite small.

The truth of this statement is demonstrated by the following investigation of some of the new data submitted by the authors, and of certain other data relating to the new 38-in. riveted steel conduit of the Rochester water-works. The Ogden steel pipe is of practically uniform diameter throughout; its length of 4 367 ft. between the two manometer stations of the authors being presumably made up of about 469 straight sections of 110 ins. each, and 27 short beveled or angle sections of about 35 ins. each, as deduced from the description of the work by Henry Goldmark,* M. Am. Soc. C. E. Furthermore, these 496 sections have a mean internal diameter of 72.22 ins., and are connected by means of butted joints with broad external round straps 10.5 ins. wide, each having four rows of rivets, so that in said length there are at least 1 984 circular rows of rivets with heads projecting inside. It also appears that there are on the average 62 rivets in each row, the mean diameter of the rivets being 1 in., and the sectional area of the conical head of such a rivet being at least 0.80 sq. in., thus presenting at each row an aggregate internal obstructing area of not less than 50 sq. ins., which is equivalent to a continuous ring or internal fillet 0.22 in. thick. The 27 angle sections, moreover, each cause an abrupt deflection of 4.5°, so that the total curvature in the line of the pipe amounts to 121.5 degrees. Account should properly also be taken of the appreciable obstruction of 11 sq. ins. of sectional area caused by the internal straps and projecting heads of the six rows of rivets of the butted longitudinal or straight seams, but it is here omitted for the sake of brevity.

Now, from the elementary principles of hydraulics, as set forth by standard authorities, the losses of head caused by interior constrictions or diaphragms in a pipe, and by abrupt angular deflections thereof, as well as by sudden enlargements of diameter, such as occur when the ends of the cylindrical sections overlap instead of being butted together, are all susceptible of computation, and it is usual to express these losses in terms of the height or head due to the mean velocity of discharge. Unfortunately the necessary empirical data available for such computations have all been derived from experiments with small pipes, but as no others are at hand, their accuracy may be tested by applying them to the cases of the Ogden and Rochester conduits; and if the results so obtained yield ultimate values for the coefficient c, in the Chezy formula $v = c\sqrt{r\,s}$, which are in close

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

accord with the values of c for smooth cast-iron or wooden pipes, it must follow that such available data are approximately correct, and also that by their use many existing uncertainties and incongruities in the discharge of riveted pipes may be eliminated. Mr. Kuichling

With this end in view, use has been made of the data afforded by experiment No. 28 of the authors, in conjunction with the other details above excerpted and deduced from Mr. Goldmark's paper. The Ogden pipe should, in the writer's opinion, be considered as practically made with four low constrictions or internal fillets (representing the equivalent sectional area occupied by the rivet heads) at every round joint, each of which produces a certain loss of head; and with reference to the notation indicated in Fig. 18, we will have the following elements:

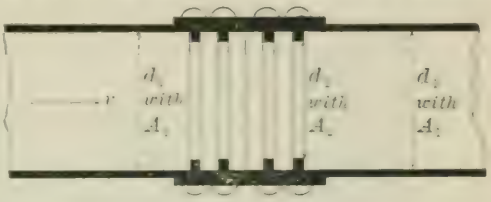


FIG. 18.

Mean internal diameter of pipe, $d_1 = 72.22$ ins.

Mean internal diameter of constriction or fillet, $d_2 = 71.78$ ins.

Ratio m of area A_2 to area A_1, $m = 0.98785$.

Number of fillets, $n_1 = 1984$.

Number of abrupt 4° 30′-deflections, $n_2 = 27$.

Length, $l = 4\,367$ ft.

Total observed loss of head, $h = 3.386$ ft.

Mean velocity of discharge, $v = 3.846$ ft.

Mean hydraulic radius, $r = 1.50458$ ft.

Observed hydraulic slope, $s = \dfrac{h}{l} = 0.00077536$.

Whence the deduced value of c, in $v = c\sqrt{r\,s}$, is $c = 112.60$. For the loss of head h_1, due to each constriction, all the standard text-books on hydraulics give Weisbach's formula:

$$h_1 = \left(\frac{A_1}{\alpha\, A_2} - 1\right)^2 \frac{r^2}{2\,g} = \left(\frac{1}{\alpha\, m} - 1\right)^2 \frac{r^2}{2\,g},$$

in which α is an empirical coefficient varying with the ratio $m = \dfrac{A_2}{A_1}$ and capable of expression within the limits $m = 0.8$ and $m = 1.0$ by approximately: $\alpha = 1.225 + 1.45\,m^2 - 1.675\,m$, as deduced from Weisbach's figures by the writer. Substituting therein the above values of m, n and r, and taking $2\,g = 64.293$ sec.-ft., corresponding to the latitude of Ogden and its elevation of 4 307 ft. above sea level, there results $\alpha = 0.98533$, and the aggregate loss of head by 1 984 constrictions $n_1\,h_1 = 0.34196$ ft.

For the loss of head due to the 27 abrupt deflections of the angle sections, each of $\varphi = 4° 30'$, we must also have recourse to Weisbach's formula:

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$$h_2 = \left(0.9457 \sin^2 \frac{\varphi}{2} + 2.047 \sin^4 \frac{\varphi}{2}\right) \frac{v^2}{2g},$$

which gives $n_2 h_2 = 0.00909$ ft.

The total computed loss of head from these two sources of resistance to the flow of water through the pipe is therefore $n_1 h_1 + n_2 h_2 = 0.35105$ ft., or over 10 % of the total observed loss of head, so that the head left to overcome the friction on the asphalt coating of the pipe is $h' = h - 0.35105 = 3.03495$ ft.; and computing therewith the value of c, so as to bring about a fair comparison with the smooth wooden pipe or with a similar line of cast-iron pipe, we obtain $c' = 118.94$; whereas the omission of the above-named losses causes this coefficient to become $c = 112.60$. Now, from the authors' experiments Nos. 40, 44, 45 and 46 on the wooden pipe, in which the velocities were nearly as great as in experiment No. 28 on the riveted steel pipe, the average value of c is $c = 125.5$, and by rejecting the highest value of c in this series the average is $c = 123.67$; hence it follows that, by properly taking into account the aforesaid losses of head in the steel conduit, which are not present in the wooden conduit, the difference between the computed values of c is greatly reduced.

It may also be noted that this small difference can readily be attributed, both to the retarding influence of the corrugations or wrinkles in the asphalt coating of the steel pipe, and to the reduction of the sectional area of the pipe by the interior longitudinal strap and rivet heads already mentioned. Such corrugations appear to be unavoidable in the usual process of coating pipes by dipping them once or twice in a bath of melted asphalt, as will be seen from Figs. 1 and 2, Plate XL, which are from photographs of the interior of one of the 38-in. and 60-in. pipes of the new Rochester conduit; and if the relatively small constrictions made by the interior rivet heads of the round seams produce a loss of more than 10% of the total head, as above computed, it cannot be doubted that the effect of the vastly larger number of smaller constrictions represented by the corrugations of the coating will become appreciable. It is therefore to be anticipated that the ultimate value of c in a steel pipe, coated in the usual manner by dipping in a bath of asphalt, and after eliminating all other special resistances as above indicated, will be somewhat less than in a smooth wooden pipe.

Proceeding next to the consideration of the first or upper section of the 38-in. Rochester conduit, the following elements are available from the construction data and the carefully made, first gauging of the discharge on October 4th, 1895, when the average age of the pipe was 1.79 years. In this case, however, all the round seams are lapped, and provided with only a single row of rivets, except for a length of about 200 ft., at every junction where the plate thickness is varied, the round seams then being made with two rows of rivets.

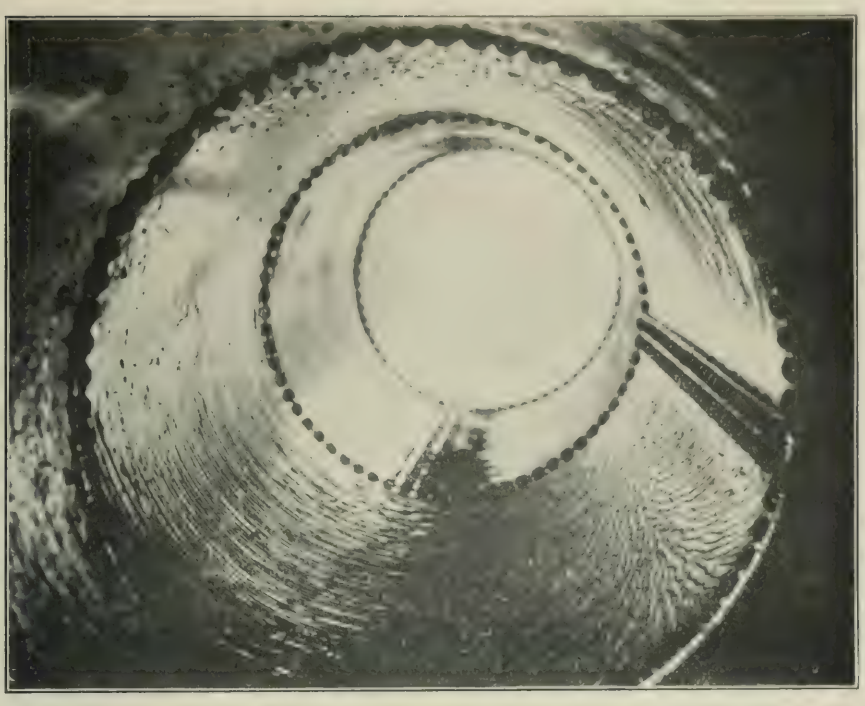

FIG. 1.

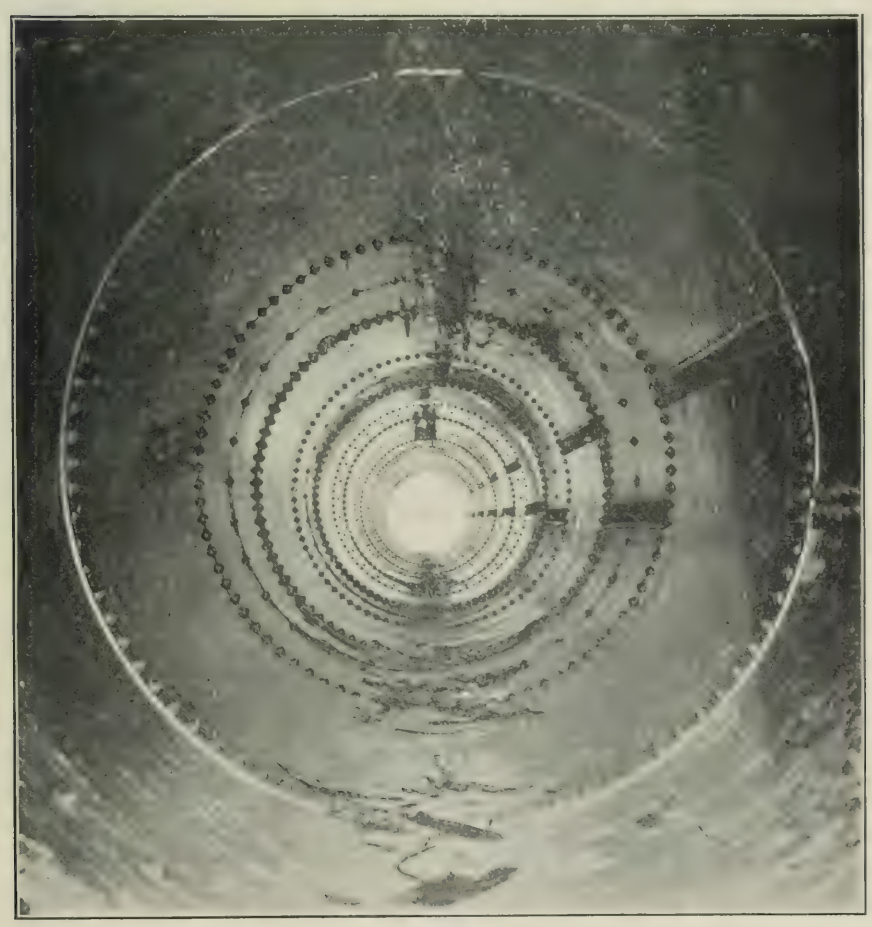

FIG. 2.

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The general style of construction, as well as the notation, is indicated in Fig. 19, and we have:

Internal diameter of inside courses or sections, $d_1 = 38$ ins.

Average plate thickness = 0.27129 in.

Average diameter of rivets = 0.6676 in.

Average sectional area of rivet head = 0.35215 sq. in.

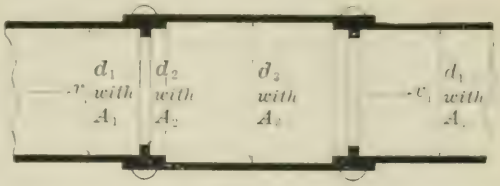

FIG. 19.

Average number of heads in round seams = 76.354.

Average area occupied by interior rivet heads in each round seam = 26.888 sq. ins.

Equivalent thickness of constriction or fillet = 0.2265 in.

Resulting constricted diameter, $d_2 = 37.547$ in.

Total number of round rows of rivets or equivalent fillets, $n_1 = 14.143$.

Average internal diameter of outside courses or sections, $d_3 = 38.5426$ ins.

Total number n_3 of abrupt enlargements of diameter from d_1 to $d_3 = 6\ 700$.

Ratio of area A_2 to area A_1, $m_1 = 0.97629$.

Ratio of area A_1 to area A_3, $m_2 = 0.97204$.

Total number of uniform deflections of $\varphi_1 = 2^\circ\ 43'\ 19''$ for horizontal and vertical curves is 914.

Same for $\phi_2 = 1^\circ\ 21'\ 51''$ is 96.

Same for average of $\phi_3 = 1^\circ\ 48'\ 48''$ for small irregular angles is 311.

Total length of pipe between points of observation, $l = 91\ 640.83$ ft., of which 87.21 ft. consists of nominally 36-in. cast-iron pipe, special castings and stop-valves, whose actual least internal diameters range, however, from 36 to 38 ins., together with one 36-in. curve having a mean radius of 12 ft., and central angle $\phi_4 = 9^\circ$.

Total observed loss of head, including height due to velocity and entrance, $h = 91.053$ ft.

Mean velocity of discharge, $v_1 = 3.2733$ ft.

Mean hydraulic radius, referred to inside courses, $r_1 = 0.79167$ ft.

Observed hydraulic slope, $s = \dfrac{h}{l} = 0.00099359$.

Whence the deduced value of c in $v_1 = c\sqrt{r_1 s}$ is $c = 116.71$.

Furthermore, for the given latitude and elevation, $2g = 64.328$.

Computing as above the losses of head due to the constrictions and abrupt angular deflections, we find: $\alpha_1 = 0.97177$;

$$n_1 h_1 = n_1 \left(\frac{1}{\alpha_1 m_1} - 1\right)^2 \frac{v_1^2}{2g} = 6.87968 \text{ ft.};$$

and the aggregate loss by the aforesaid deflections, as well as the

Mr. Kuichling. cast-iron curve, $\Sigma h_2 = 0.09744$ ft. For the loss of head by the abrupt enlargements of diameter from d_1 to d_3, the usual formula is:

$$h_3 = \left(1 - \frac{A_1}{A_3}\right)^2 \frac{v_1^2}{2g} = (1 - m_2)^2 \frac{r_1^2}{2g},$$

from which there follows: $n_3 h_3 = 0.87222$ ft.; and finally, for the velocity head and the loss of head caused by the passage through ten stop-valves and special castings, we have respectively, 0.16656 ft., and 0.13910 ft. The sum of all these losses of head is 8.155 ft., which being subtracted from the total observed head or fall, leaves for the available head to overcome the friction on the asphalt coating: $h^1 = h - 8.155 = 82.898$ ft. With this fall the corrected value of the coefficient c for the steel pipe, when divested of the several special resistances above enumerated, becomes $c^1 = 122.32$. Now, from two equally careful gaugings made on October 17th and November 7th, 1895, of the second section of the Rochester conduit, whose upper part consists of 924 ft. of 36-in. cast-iron pipe, the values of the coefficient c in the latter for velocities of 4.204 and 4.234 ft. per second were found to be, respectively, 129.45 and 125.25, the loss of head in a length of 889.57 ft. of said pipe being determined separately from open piezometers; hence in this case also, the difference between the values of c for cast-iron and riveted steel pipes becomes quite small when proper allowance is made for the losses of head peculiar to the latter kind of pipe.

It may also be mentioned that the above mode of computation was applied by the writer to a few other cases of new riveted pipe where accurate data have recently become available, with results similar to those above set forth; and he is therefore inclined to believe that the empirical coefficients deduced by Weisbach from experiments with constrictions in small pipes are approximately applicable to large pipes. At all events, the foregoing computations relating to the 6-ft. Ogden pipe and the 38-in. Rochester conduit, appear to confirm this view quite strongly. Experienced hydraulicians will, however, prefer to make use of coefficients derived from experiments on a large scale; and, as the subject is of great commercial importance, it is hoped that suitable investigations of the losses of head due to rivet heads, constrictions, deflections and abrupt enlargements, as indicated above, will soon be undertaken and published. Engineers will then be able to make much more intelligent analyses of complex hydraulic problems, and less room for error will occur in the choice of a proper "coefficient of roughness" for pipes. In fact, it is manifestly unfair to make use of such a coefficient except in cases where exactly similar conditions prevail, or where the influence of special disturbances has been eliminated, as was done in the preceding instances of new constructions. After incrustations of rust, sediment, organic growths, etc., have made their appearance on the interior of a pipe, other con

ditions arise which are probably altogether too complicated to admit of initial computation, and in such cases the results of practical experience alone must govern. Mr. Knichlin

In illustration of these latter remarks, the writer cites the results of several subsequent gaugings of the 36-in. cast-iron pipe in the second section of the Rochester conduit, referred to above. As already stated, the values of the coefficient c found from the observations of October 17th and November 7th, 1895, when the pipe was still comparatively new, were, respectively, 129.45 and 125.25; while from the tests made on July 28th, November 11th and November 19th, 1897, and on June 4th, 1898, said coefficient was found to have fallen to 99.22, 66.84, 66.24 and 70.23, respectively; the cause of the great decrease probably being the profuse development of organic growths in the pipe at the intermediate storage reservoir, where both the pressure and the normal velocity of flow are quite small. In contrast with these values of c for the originally smooth and clean cast-iron pipe may be placed the corresponding values of c for the succeeding long line of 38-in. riveted steel pipe, which were, respectively, on the dates mentioned: 109.35, 109.07, 106.11, 107.11, 106.46, and 105.65. It should also be noted that these figures refer only to the second section of the conduit, in which the conditions differ somewhat from those in the first section previously considered.

In connection with the various resistances to motion in lap-jointed steel pipes, mention may likewise be made of the loss of head caused by the passage of the water from the "outside" to the "inside" courses, or from the section A_2 to A_1, Fig. 19. No computation of this loss was made by the writer, inasmuch as reliable empirical data therefor, with small differences of A_2 and A_1, are not yet available, also because such loss appears to be insignificant in comparison with the loss due to enlargement of section, or the passage from A_1 to A_2; and it is also probable that such loss is compensated by the slight advantage of flow gained in the larger sectional areas of the outside courses. Another important element is the deformation or flattening of a large riveted pipe made of thin plates, by the weight of the earth covering, in places where the internal water pressure is too small to insure a full cylindrical cross-section. While this point does not apply to the Ogden pipe, where the pressures are very great, it is nevertheless pertinent to several other riveted conduits cited by the authors, and is of appreciable influence in the resulting values of the coefficient c. Such a flattening produces a reduction of sectional area while the wetted perimeter remains constant, and hence causes a decrease in the value of the mean hydraulic radius r, which, if duly taken into account, increases the value of c, as shown by the following example:

The paper on "The Distortion of Riveted Pipe by Back-Filling," by D. D. Clarke, M. Am. Soc. C. E.,* cites the case of a new 42-in. steel

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 93.

Mr. Kuichling. pipe, whose original vertical diameter was reduced at some points about 4 ins. by the weight of the superincumbent earth. This reduction is nearly 10% of the initial diameter, and produces an appreciably smaller sectional area. To investigate the case generally, let it be assumed that the flattened circle becomes an ellipse with an area A_1, a constant circumference $\pi\, d_0$ and a vertical minor axis d_1, the diameter of the original circle being d_0 and its area A_0; also let $n = \frac{d_1}{d_0}$ and $m = \frac{A_1}{A_0}$.

Then

$$m = \frac{n}{5}(4 - 3\,n)\left\{1 + \sqrt{1 + \frac{n\,(40 - 25\,n)}{(4 - 3\,n)^2}}\right\}.$$

Furthermore, for any given discharge Q through the pipe,

$$Q = A_0\, v_0 = A_0\, c_0 \sqrt{r_0\, s}, \text{ or } Q = A_1\, v_1 = A_1\, c_1 \sqrt{r_1\, s},$$

according as the section is a circle or an ellipse; and since

$$r_0 = \frac{A_0}{\pi\, d_0} \text{ and } r_1 = \frac{A_1}{\pi\, d_0}, \text{ or } \frac{r_1}{r_0} = \frac{A_1}{A_0} = m,$$

there follows:

$$\frac{c_1}{c_0} = \frac{A_0 \sqrt{r_0}}{A_1 \sqrt{r_1}} = \frac{1}{m \sqrt{m}}, \text{ or } c_1 = \frac{1}{m^{\frac{3}{2}}} c_0.$$

Now, for a reduction of 10% in vertical diameter, or for $d_1 = 0.9\, d_0$, $n = 0.9$ and $m = 0.9857$, whence $\frac{1}{m^{\frac{3}{2}}} = 1.022$ and $c_1 = 1.022\, c_0$; thus showing that for such a deformation or flattening of the pipe the coefficient c_1 becomes 2.2% greater than for the truly circular pipe.

Doubtless the foregoing refinements of computation will appear trivial to many engineers who are accustomed to dealing with ample margins of safety in the design of public works, but they become necessary when comparisons are made, either with the view of establishing improved data for future guidance, or to arrive at fundamental principles, such as the proper value of the coefficient c for different classes of surface.

In conclusion, the writer desires to avail himself of this opportunity to correct a numerical error which was inadvertently allowed to appear in one of his published reports,* and which was quoted by the authors in their paper at page 504. The experiments referred to were made in June and October, 1896, on the lower half of the first section of the new Rochester conduit, the length of 38-in. riveted steel pipe here involved being 46 338.76 ft., or 8.776 miles, instead of 34 468 ft., or 6.53 miles, while the total fall or loss of head, as computed from manometric observations at the lower end, ranged from 0.922 to 8.391 ft., instead of 1.08 to 6.95 ft. For the whole series of experiments,

* Report of Executive Board of Rochester, N. Y., for 1896.

the value of c ranged from 69.496 to 112.154, with an average of 101.78 instead of 99.04; but by excluding the three lowest values of c, which relate to the three smallest velocities, namely, $v = 0.276$, 0.489 and 0.612 ft. per second, the average value becomes $c = 107.35$; furthermore, by taking into account only the five experiments which were of longest duration and wherein the velocities ranged from 0.758 to 1.223 ft. per second, the average value of c becomes $c = 105.12$. Mr. Kuichling

It should be noted that this series of gaugings was made to find the discharge through the said portion of the conduit for different degrees of opening of the 8-in. by-pass valve on a 36-in. stop-valve located at the upper end of said portion, the large valve remaining closed. The various discharges and velocities in the conduit were computed from the observed rise of the water in the large storage reservoir at the lower end of said portion during periods of from 4.2 to 13.6 hours, with the addition thereto of both the observed evaporation from the water surface during such periods and the previously measured rate of percolation or leakage of the reservoir when all the inlet and outlet gates thereof were tightly closed, so as to obtain the gross delivery of the pipe. The greatest computed velocity, 1.223 ft. per second, occurred when the said 8-in. by-pass valve was wide open; and the resulting average values of c above mentioned may be compared with the following values of c for the entire length of 91 640.83 ft. of the first section of the conduit, as found at various times for full flow under existing conditions of restricted discharge at the reservoir: October 4th, 1895, $v = 3.2733$ ft. per second, and $c = 116.71$; December 23d, 1895, $v = 3.2308$ and $c = 114.05$; July 23d, 1897, $v = 3.2482$ and $c = 115.50$; November 18th, 1897, $v = 3.1481$ and $c = 112.04$.

To complete this correction of error in the report mentioned, it should also be added that the mean value of the coefficient c, there stated to have been found in 1895 for the entire first section of the conduit, should be $c = 115.38$ instead of 109.25, which latter refers to the second section. The details of these experiments were not published, owing to the discordances in the resulting values of c, which were apparently due to slight errors of observation with the mercury manometer, as in the case of the Ogden experiments; also, because it was intended to repeat the observations with the utmost care, and extend them over longer periods of time.

CORRESPONDENCE.

Mr. Adams. ARTHUR L. ADAMS, M. Am. Soc. C. E.—The great disparity in the results obtained from the experiments on the steel pipe, especially for velocities of less than 2 ft. per second, are discouraging to any attempt at analytical study. More particularly is this true because of the unlooked-for difficulty with the mercury gauge having cast some doubt upon the reliability of the results.

The figures obtained from the stave pipe experiments are much more harmonious for velocities exceeding 1½ ft. per second. It would seem that a much greater length of pipe than 2 700 ft. might have been used to good advantage, especially since a direct measurement of heads could not be made by water piezometers, there being no better way of eliminating the effect of the usual uncertainties arising in the reading of the gauges and from other causes. The writer always feels disposed to give weight to such experiments, when conducted with reasonable care, nearly in proportion to the length of the line experimented on.

The results, as a whole, are, in a measure, surprising. Most recent experiments have tended to cause the lowering of previously conceived ideas of the probable values for c in the formula $v = c\sqrt{r\,s}$ when applied to single-lap riveted pipes of large diameter. Indeed engineers are probably quite prepared for the receipt of further evidence that the value of c in pipes of this type is nearly independent of the diameter and also of the velocity when the latter exceeds about 2½ ft. per second; but, that a smooth cylindrical pipe without any abrupt deflections or internal projections, such as the stave pipe experimented upon, should give no greater values for c than those presented (values, by the way, which are considerably less than have frequently been obtained with clean cast-iron pipes of large diameter and from stave pipes of much smaller diameter), it is believed will scarcely be in accord with the previously conceived opinions of many engineers.

While the number of actual experiments heretofore made to determine values of c suitable for stave pipes are exceedingly limited, as has been pointed out by the author, yet the numerous experiments on open and closed channels having uniformity in section and in condition of inner surface as to roughness, though, perhaps, to a degree less perfect than usually exists in a stave pipe, have, in almost all cases, within the limits of customary use, shown an increase in the value of c with each corresponding increase in the value of r. The experiments described by the authors do not indicate that such a law of increase applies to pipe as large as 6 ft. in diameter.

The results will, doubtless, be of value in finally determining a formula which shall be better adapted to pipes of this class than any at present used, but the writer feels that final judgment in this matter

Mr. Adams.

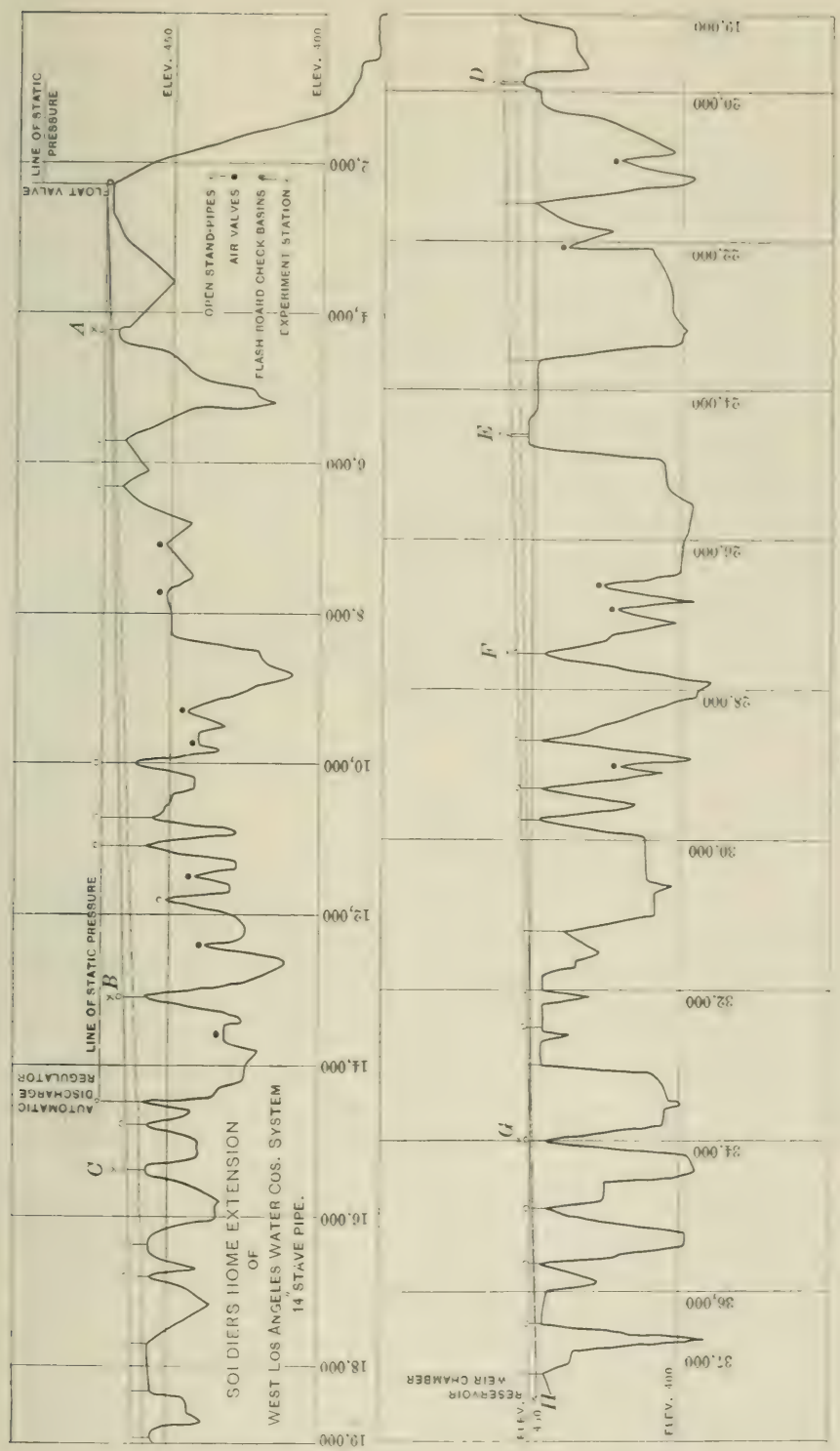

FIG. 20.

Mr. Adams. must be suspended until many additional experimental data have been accumulated. The full value of any experiments of this class yet conducted, since they are so comparatively few in number, cannot be ascertained until then.

As a slight contribution to such knowledge, the writer presents herewith the results of some experiments on a line of 14-in., redwood, stave pipe recently constructed under his supervision by the West Los Angeles Water Company for the supply of the Pacific Branch of the National Soldiers' Home.

In preparing for these observations the average diameter of the pipe was carefully determined by making many measurements of the circumference of the completed structure. These measurements covered all degrees of band spacing, which varied from $4\frac{1}{2}$ ins. to 10 ins. The staves were uniformly $1\frac{9}{64}$ ins. in thickness. The net diameter varied from 13.99 ins. to 14.11 ins. By reference to the profile, Fig. 20, it will be seen that the vertical curves are quite numerous, but they are all made without the use of specials and with radii of not less than perhaps 40 ft. The horizontal curves are few, and 286 ft. was the minimum length of radius used.

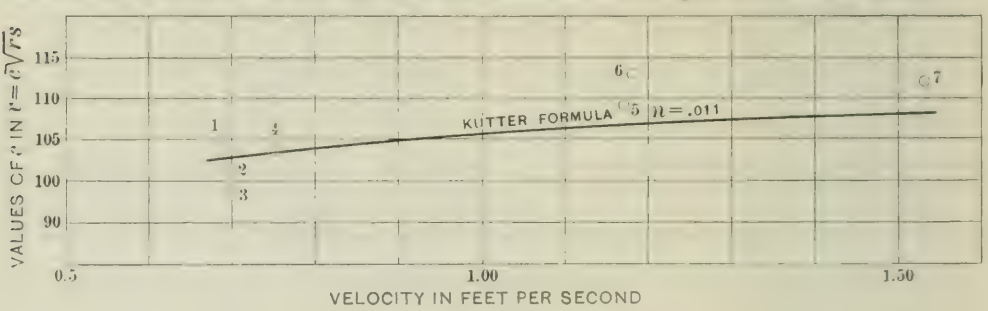

Fig. 21.

In these experiments the discharge was measured by means of a 4-ft. weir located at the terminus of the line; the head being measured by means of a hook gauge. The customary weir formula $Q = CL\sqrt{h^3}$ was used. In the fifth experiment a volumetric measurement of several hours' flow into a small concrete reservoir, 8 ft. deep and 50 ft. in diameter, with vertical inside walls, indicated 3.33 as being the correct value for C. The same value was used in computing the discharge in the other experiments. The loss of head was observed by noting the position of the water in the open stand-pipes and other designated structures. The effect of the slight fluctuations in the stand-pipes was eliminated by taking the mean of many readings, a suitable length of time having elapsed after the beginning of the experiment. The relative elevations of different points of observation, excepting A and B, were determined by a wye level. The points of uniform height at A and B were determined by closing a stop-gate at station 145, and allowing the water to come to rest. The outgoing pipe from the structure at D is enlarged to 16 ins. at the outlet. An allow-

ance for the velocity head only was here made. With the low velocities used, no sensible error could have thus resulted. Mr. Adams.

These experiments, classified as *A*, *B* and *C* in the order of their probable merit, are as follows:

TABLE No. 11.

No.	Section Experimented on.	Length of section-feet.	Velocity in feet per second.	Average pipe. Diameter. Inches.	Loss of head in 1 000 ft.	Resulting values of c in $v = c\sqrt{rs}$	Relative value of experiments.
1	2	3	4	5	6	7	8
1...	$C - D$	4 403	0.691	14.11	.145	105	*A*
2...	$G - H$	3 436	0.691	14.08	.161	101	*B*
3...	$D - E$	4 825	0.698	14.05	.170	99	*C*
4...	$A - B$	8 931	0.751	14.05	.178	104	*B*
5...	$F - H$	9 912	1.167	14.05	.391	109	*B*
6...	$A - B$	8 931	1.181	14.05	.375	113	*A*
7...	$F - H$	9 912	1.531	14.05	.638	112	*A*

It will be noticed that these experiments have been made on five different stretches of pipe, ranging in length from 3 436 ft. to 9 912 ft.; and that only low velocities could be secured, such as usually produce the most discordant experimental results. Considering these conditions it would appear that the resulting values of c are quite concordant.

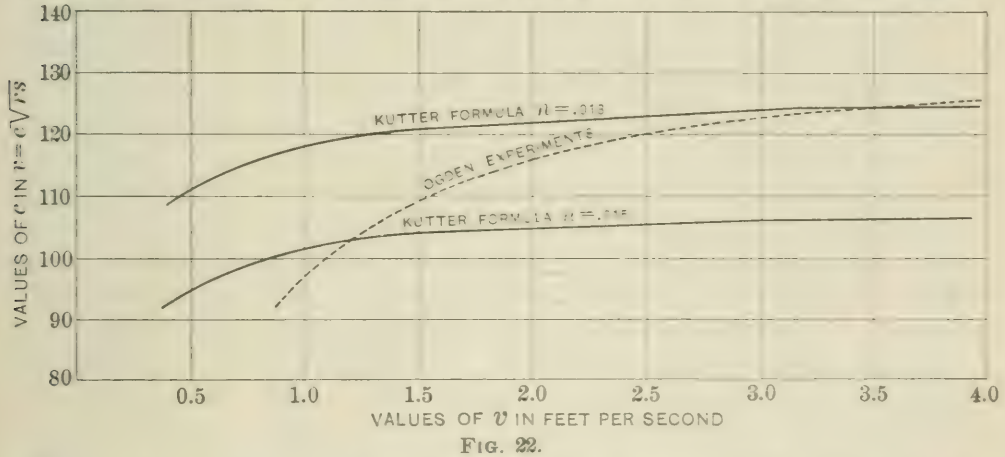

FIG. 22.

These values, while conforming quite closely to results obtained by the use of the Kutter formula, if n be taken at from .0108 to .011, for these velocities do not indicate .01 as being a safe assumed value for n, as have all previous experiments mentioned by the authors. Could higher velocities have been obtained, the results might then have shown that a lower value for n would have best fitted the results. This is indicated by the position of the observed values of c in relation to a curve given by the Kutter formula (Fig. 21). The same tendency of the experimental curve to depart from the Kutter curve in the low velocities is shown to exist in the Ogden experiments in Fig. 22.

Mr. Noble. T. A. Noble, M. Am. Soc. C. E.—The general conclusions arrived at by the authors, that the value of the coefficient c in the formula $v = c\sqrt{r\,s}$ is much less for wood pipes of the size experimented upon than had heretofore been supposed, and that the average value of n in Kutter's general formula for the flow of water in channels is about .013, will come as a surprise to the engineering public. The difference in the carrying capacity of wooden pipe, figured from this coefficient would be great enough to seriously affect the capacity of a number of large conduits already constructed. It is, therefore, a matter of considerable importance that this question should be settled by a more extended series of experiments under, perhaps, more favorable conditions.

Several points which might have affected the results have occurred to the writer, and these are given with the object of getting the benefit of the authors' comments thereon.

It would seem that the distance between the points of observation on the wooden pipe was too short to obtain accurate results for the low friction heads and rates of discharge within the range possible in these tests. The greatest difference in head, in feet of mercury, is only 0.115. Any error due to reading the manometers or from any other cause would have a very much greater effect where the difference, as in this case, is small.

In the profile of the pipe line, as given in Mr. Goldmark's paper,* it will be noticed that there are about twenty-eight high points between the point of entrance and the point of discharge, whereas only sixteen air valves are shown. If the valves do not discharge air under pressure, they would be of no value for this purpose except when attended by some one detailed to open them frequently enough to keep the air from accumulating at these points. Is it not possible that during these tests a considerable accumulation of air was present in the pipe, in sufficient quantity to affect the rate of discharge? It would be germane to know if ample provision had been made to keep the pipes free from air at every point where it would be possible for it to accumulate.

There is apparently no uniformity in the relation between the average velocity in the pipe and the friction head, as determined by these experiments, except that, in general, the head increases with the increase in velocity. It is, therefore, doubtful if any conclusion as to the value of c and n in Kutter's formula, in any larger rate of discharge would be reliable. Table No. 12 gives the values of n for the different values of r, s and c, as determined from these experiments.

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

TABLE No. 12. Mr. Noble.

Number of test.	v.	s.	c.	n.
35	.543	.0000401	69	.0204
47	.544	.0000251	88	.0160
41	.666	.0000551	73	.0199
31	.816	.0000201	148	.0100
48	1.217	.0000953	101	.0151
36	1.228	.000103	100	.0153
37	1.242	.0000952	104	.0148
42	1 332	.000106	106	.0145
31	1.482	.000111	115	.0135
49	1.876	.000181	114	.0138
43	1.978	.000211	111	.0141
32	2 138	.000211	120	.0132
38	2.218	.000235	117	.0135
50	2.536	.000300	119	.0134
33	2.689	.000331	120	.0133
39	2.783	.000361	119	.0134
51	3.149	.000452	121	.0132
34	3.453	.000517	124	.0129
40	3.568	.000566	122	.0132
46	3.617	.000501	131	.0123
45	3.627	.000547	126	.0127
44	3.645	.000576	123	.0130

There seems to be a decrease in the value of n with the increase in velocity. If this be true, it is possible that, when the pipe is discharging, up to the capacity for which it is designed, the value of n will be much less than that stated.

The change in the rate of discharge, which is constantly going on in a plant where power is distributed to consumers and where the load is constantly changing, must have a serious influence upon the apparent friction head, between any two points on the pipe line. A decrease in the rate of discharge would increase the head at the point of discharge. This effect becomes less, the greater the distance from the point of discharge, until it is zero at the point of entrance. The effect of an increase in the rate of discharge would have a corresponding but opposite effect upon the apparent friction head. Where a sufficient number of observations are taken it would seem that the errors from this cause would be neutralized. This, however, is not known, and hence the nature and value of this effect upon the friction head during these tests is unknown. Is this not the probable cause of the fluctuation in the relative value of v and s from which c (and n in Kutter's formula) are determined ? In any case, it seems evident that, to get reliable and consistent results, the rate of discharge during each observation must be constant. Any acceleration or retardation of the rate of discharge would have a serious effect upon the friction head. This would be particularly true if the effect of retardation is not equal to the effect of acceleration, and if the rate of discharge at the end of the experiment were not equal to the rate at the beginning.

Mr. Weston. EDMUND B. WESTON, M. Am. Soc. C. E.—In 1889 the writer presented to the Society a paper* in which he gave his reason for considering Darcy's formula for the flow of water in pipes a very valuable formula for calculating the loss of head in pipes having interior surfaces similar to new cast-iron pipes; and he was much pleased in reading this interesting and valuable paper to note the small difference between 0.0216, the average experimental value of f obtained by the authors, as given in the paper, and 0.0202, which is a corresponding value of f deduced from Darcy's formula (reduced to English measures by the writer), for a pipe 72 ins. in diameter. The difference between the two is about 7 per cent. The authors state on page 498, that it was thought that the method adopted for measuring the loss of head should give results reliable to within 0.1 ft. in absolute value. The average of the losses of head given in column 8 of Table No. 2 is 1.342 ft., and 0.1 is about 7.5% of 1.342; whereas, the difference between the f obtained by the authors and that computed from Darcy's formula is, as given above, about 7 per cent. The writer recognizes the fact that the interior surface of a new, riveted steel pipe is more or less different from that of a new cast-iron pipe, and should expect the values of f for the two kinds of pipe to vary slightly, and, when computing results for steel pipes, he has been in the habit of increasing the value of f, deduced from Darcy's formula, by a small percentage, dependent in amount upon the character of the steel pipe and its joints. These percentages the writer determined from the results of a number of experiments made with steel pipes by different authorities, and he will still continue to make use of them, although the results given in the paper, considering the possible error mentioned by the authors, tends to show that Darcy's formula could have been advantageously used for determining the size of a pipe having similar characteristics to the 72-in. steel pipe used by the authors in making their experiments. For instance, on pages 492 and 493, the highest velocity given is 3.846 (No. 28), and the corresponding experimental loss of head per 1 000 ft. is 0.776. The loss of head per 1 000 ft. computed by Darcy's formula (as deduced by the writer), using the same velocity, is 0.774 ft., which is 0.002 ft. less than the experimental result. Or, again, calculating the loss of head by using 0.0216, the average value of f obtained by the authors, the result is a loss of head of 0.828 ft. per 1 000 ft., which is 0.054 ft. or about 7% more than that obtained by Darcy's formula; but within the limit of the possible error 0.1 mentioned by the authors on page 498.

Mr. Le Conte. L. J. LE CONTE, M. Am. Soc. C. E.—This paper is a valuable and timely contribution. The velocity coefficients for the 72-in. riveted pipe are somewhat less than many engineers would be willing to assume; yet they are such as every conservative engineer would natu-

* *Transactions*, Am. Soc. C. E., Vol. xxii, Jan., 1890.

rally anticipate, basing his judgment on reliable data obtained from careful experiments with smaller pipes. Mr. Le Conte.

Riveted pipes have been extensively used on the Pacific Coast since 1870 – the sizes running all the way up to 44 ins. in diameter and the thickness of the plate-iron from No. 14 (B. W. G.) up to $\frac{3}{8}$ in., while the velocity of flow varied from 1 ft. up to 10 ft. per second.

The general results obtained by practical experience go to show the paramount importance of the influence of the thickness of the plate-iron used. This factor is so important that the coefficient determined by experience on one pipe line is of no value whatever as applied to another line having the same diameter and slope. Hence it follows that the proper steps to be taken in any case, in order to select the most rational value for c, involves much more laborious calculation than is the case with any other class of pipe. With a riveted pipe line the velocity coefficient c changes materially with every change in the thickness of the iron, and before a satisfactory solution of the problem can be obtained the number of linear feet of No. 14 pipe, No. 12 pipe, No. 10 pipe, No. 8 pipe, No. 6 pipe, No. 4 pipe, etc., must be determined. This can best be taken from the profiles of the pipe line. Then, in the same manner as in the summation of moments, a value for the grand mean thickness of all the plate-iron used throughout the entire pipe-line can be deduced. This being completed, reference should be had to the Table of Coefficients (Table No. 13) submitted by the writer, and under the computed mean thickness and opposite the proper diameter may be found the most rational or true mean coefficient c, peculiar to the particular pipe line being considered.

It follows from the above that it is quite rare to find any two riveted pipe lines of the same diameter and slope which actually develop the same value for the velocity coefficient, because each pipe-line has its own particular value for c. Table No. 13 has been prepared by the writer for the purpose of facilitating the selection of a rational velocity coefficient for riveted pipe lines, using the old standard formula:

$$v = c\sqrt{r\,s}.$$

TABLE No. 13.—Coefficients Showing the Changes due to Different Diameters and Thicknesses of Iron in Riveted Pipe Lines.

Thickness—B. W. G.

Diameter in inches.	No. 16.	No. 14.	No. 12.	No. 10.	No. 9.	No. 8.	No. 7.	No. 6.	No. 4.	$\frac{1}{4}$ in.	No. 3.	$\frac{3}{8}$ in.
44	.....	.....	.....	120	119	117	116	114	112	111	110	100
42	.....	.....	.....	116	115	114	113	111	109	109	108	100
37½	.....	116	114	112	111	110	109	108	105	104	104	95
30	.....	114	112	110	109	108	106	105	102	101	100	91
24	.....	111	109	107	106	104	103	101	98	97	97	87
12	100	99	97	95	94	93	91	90	88	87	86	78

Mr. Le Conte. The authors refer to Table No. 4, and call attention to Kutter's sub-coefficient (n) increasing with the diameter. The writer would simply invite the authors' attention to the table of coefficients, herewith submitted, as redundant evidence of the fact that the coefficient (n) is a particularly active function in the case of riveted pipe lines, and that, unless full and complete information is furnished showing the extent and thickness of each and all the plate-iron used in manufacture, it is simply impossible to draw comparisons which have any value whatever.

The portion of this paper which is particularly interesting to the writer is that describing the experiments on the 72-in. wooden pipe, wherein the coefficient c varies from 100 to 126. The facts presented constitute a valuable addition to the knowledge on wooden pipe capacity and will prove to be a shock to many active and prurient advocates who have been claiming a safe and reliable value for $c = 166$, to be followed by an annual increase in value due to the "sliming up" of the interior surface of the pipe.

As to the changes in the value of the coefficient c with changes in the velocity of flow, experience on the Pacific coast seems to show that, for the larger sized pipes, the changes, if any, are very slight, but in the smaller sizes, especially when made of heavy plate-iron, the velocity coefficient c diminishes very materially with each increment of velocity of flow up to 10 ft. per second.

Mr. Henny. D. C. Henny, M. Am. Soc. C. E.—The authors' experiments on the carrying capacity of these large wooden and steel pipes are of the highest interest. If they may be accepted as fairly correct it seems that the ultimate power-producing and earning capacity of the plant as a whole has been seriously overestimated. This is shown by a comparison between the values of c, as assumed by the designers, and those ascertained by the experiments of the authors. While the highest velocity experimented with was less than half of that on which the estimate of the ultimate capacity was based (8.75 ft. per second), the curves shown on Figs. 7 and 11 permit a rough approximate estimate of the values of c for said maximum velocity, which for steel pipe would be about 111 and for wooden pipe 130. As the respective values assumed by the designers were 120 and 160, it is evident that a deficiency of about 8% for steel pipe and of about 19% for wooden pipe may be found.

As far as wooden stave pipe is concerned, the adoption of the value $c = 160$ appears to have been justifiable. It is true that, owing to the comparatively recent introduction of this class of pipe, data which were directly available were almost entirely lacking, yet all the knowledge which existed pointed toward a value of c as high, or even higher, than that assumed.

In addition to the recent experiments quoted by the authors, there are available the results of three experiments of the Darcy & Bazin

series, Nos. 24, 25 and 26, which are fairly applicable, although they were made with open semi-circular channels and not with closed pipe. Where the depth in the former equals the radius there occur to the writer only a few points of difference between them and the closed pipes. Mr. Henny

First.—The resistance of the air in contact with the water surface in semi-circular channels.

Second.—The possible occurrence of vertical curvature in the pipe.

Third.—The greater uncertainty regarding the actual path of each water particle in the pipe, the possible deviation from the hypothetical path being twice as much in a vertical direction as in the flume.

The first point of difference is in favor of the pipe, the other points are in favor of the flume, and all, so far as known, are small in ordinary cases.

The following table is offered as facilitating comparison:

TABLE No. 14.

Authority.	Author's No. of series.	Character of surface.	Form of cross-section.	Diameter in inches.	Fall per 1 000.	Observed velocity. Feet per second.	Coefficient c, Chezy form.	Coefficient n, Kutter form.
Adams.....	....	Planed wood.	Circle.	18	1.9628	3.605	132.9	.0099
Schuyler....	....	Planed wood.	Circle.	30	.....	.....	.....	.0096
Darcy and Bazin.	24	Pure cement.	Semi-circle open at top. Depth of water-radius.	49.20	1.50	6.06	154.2	.0102
	25	$\frac{2}{3}$ cement. $\frac{1}{3}$ fine sand.		49.20	1.50	5.55	141.7	.0109
	26	Partly planed wood.		55.08	1.50	5.54	133.5	.0118
Stearns...	....	Coated cast iron, new.	Circle.	48	0.318	2.616	146.7	.0105
	....		.....	.....	0.711	3.738	140.1	.0110
	....		.....	.....	1.221	4.965	142.1	.0109
	....		.....	.....	1.849	6.195	144.1	.0108
Fitz-Gerald.	34	Same as above, 16 years old, tubercles removed.	Circle.	48	0.0689	1.113	134.1	.0107
	32		.....	.....	0.3234	2.472	137.5	.0111
	46		.....	.....	0.7167	3.723	139.1	.0111
	48		.....	.....	1.2414	4.973	141.1	.0110
	44		.....	.....	1.8283	6.141	143.6	.0108
Marx, Wing and Hoskins.	37	Planed wood.	Circle.	72.50	0.0952	1.242	104	.0148
	49		.....	.....	0.181	1.876	114	.0139
	33		.....	.....	0.331	2.689	120	.0134
	34		.....	.....	0.517	3.453	124	.0131

NOTE.—The serial numbers selected from the FitzGerald and the Marx, Wing and Hoskins experiments are representative of their respective series, and were chosen for their close coincidence, when platted, with the average curves presented by the experimenters.

Considering that planed wood offers less resistance to flow than any of the other surfaces mentioned, a comparison on the basis of the Chezy formula shows that the values of c obtained for the Ogden wooden pipe are remarkably low. From the Darcy and Bazin, Stearns

Mr. Henny. and FitzGerald experiments it would be concluded that, for a velocity of 6 ft. per second in a 4-ft. wooden-stave pipe, a value of $c = 155$ might well be expected, and in view of the tendency of c to increase, both with the diameter and the velocity, the designers in assuming a value of $c = 160$ for a 6-ft. wooden pipe, and 8.75 ft. per second velocity, showed no particular lack of conservatism.

The previous experiments with 18-in. and 30-in. wooden-stave pipe afford a less direct means of comparison on the basis of the Chezy formula owing to the great difference in diameter. If comparison is made on the basis of the Kutter formula it is found that even the lowest value of n deduced from the experiments on wooden-stave pipe made by the authors is much higher than it was reasonable to expect. The authors have, however, called attention to the fact that their results, both with wooden and steel pipe, lead them to question the reliability of this formula for large pipe. While it must be admitted that the variation in the values of n for varying velocities is large, yet the unquestioned wide applicability of the Kutter formula to open waterways of all dimensions and the fairly satisfactory results obtained, in applying it to pipes with diameters up to 48 ins., appear to necessitate stronger evidence than has so far been adduced before discarding its use in connection with diameters above 48 ins., although results such as have been obtained with the Ogden pipe may compel the allowance of greater margins of safety.

That the results obtained were not in accord with previous experience must have become apparent to the authors at an early stage of their experiments, and must have had the effect of rendering them more than usually cautious in guarding against errors in every direction. If the writer, therefore, further on, touches upon some elementary factors entering into the problem, it is with the expectation that the authors are fully prepared to furnish such additional information and data as may be deemed necessary to remove all reasonable doubt as to the thorough reliability of the results.

The authors have set forth with admirable clearness certain elements affecting the accuracy of their work. As is shown by them, the values determined from low velocities, say below 1 ft. per second, are unreliable. Comparing the values found in the wooden pipe experiments, for velocities above 1 ft. per second, among themselves, it is found that the variations from the average amount to about 6% both ways. Variations similarly determined from the FitzGerald experiments on 48-in. clean cast-iron pipe* amount to 1.3%, the length of pipe experimented with, expressed in diameters, being about the same and the range of velocities being greater. The unfavorable condition under which the authors labored, in respect to the drop of the grade line at both the upper and lower station with

* *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 261.

each increase of velocity, probably contributed to this result. The effect of this would have been less serious had the upper station been nearer the intake and had the section itself been longer. This is so evident that it may be presumed that serious obstacles prevented the use of the long upper section of the pipe above the tunnel for experimental purposes. Aside from the errors resulting from inaccuracies in observation, it may be well to consider possible errors not mentioned by the authors and affecting all observations alike. Mr. Henny.

The measurement of the length of pipe between the upper and lower stations leaves probably nothing to be desired. The determination of the average diameter must, however, have been a much more difficult matter, as the pipe was buried for almost its entire length. It has not been stated how the value, $72\frac{1}{2}$ ins., has been ascertained. From the dimensions of the bands and staves, as given in Mr. Goldmark's paper,* the writer would have concluded, in the absence of actual measurements after completion, that the diameter would most likely lie between $72\frac{1}{4}$ and 72 ins. The designers themselves, in computing the capacity, assumed the diameter at 6.03 ft., or about $\frac{1}{8}$ in. less than the authors. The length of the bands and the available thread permit a reduction of diameter to $71\frac{3}{4}$ ins., in cinching, which would be a reduction in circumference of less than 0.9%, starting from $72\frac{3}{8}$ ins., a larger proportionate reduction having been frequently measured by the writer on other wooden-stave pipe, particularly where the spacing of the bands is as close as on the pipe under discussion. This computed minimum diameter for dry pipe may suffer an additional reduction from the swelling of the staves which meets with resistance in every direction except toward the center of the pipe. Unless the interior diameter has been derived from actual measurements the possible error from this source exceeds 2 per cent.

The presence of air at the summits may have a seriously disturbing effect on the grade line, and has probably been carefully guarded against. The relief shaft at tunnel No. 7, if connecting with the tunnel at its top, may be expected to free the water of nearly all air. Yet, wherever, a summit occurs there exists danger of air accumulation, and in view of the presence, between the upper and lower stations of three summits and one practical summit at station 250, some positive evidence of absence of air, at these points, is required to allay doubt.

The omission, from the profile given by Mr. Goldmark,† of air-valves at some of these points may have been accidental, except possibly at station 250, but it should further be noticed that such air-valves as are described are not automatic, and remain closed while under pressure. In the account of the steel pipe experiments mention was made of careful manipulation being needed to keep the long pipe

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

Mr. Henny. connecting the steel main with the manometer at the lower station wholly free from air. Did this indicate the presence of air in the water?

The writer has been informed that the Ogden River, during certain seasons of the year, carries a considerable proportion of sand. This, taken in connection with the absence of a reservoir at the upper end, implies the danger of sand having been carried into the pipe, unless a thoroughly effective sand trap was provided. The maximum velocity in the pipe used in experimenting was capable of transporting sand for long distances, and all the circumstances, unless wrongly understood by the writer, point toward the possibility of the presence of sediment in portions of the pipe line. Nor could the 6-in. gates which occur at depressions be in any way depended upon to flush such sediment out except for a few feet either way from the points where they are attached. In first opening they would enable some idea to be formed regarding the presence or absence of sediment in the pipe, otherwise they can be useful only as drain gates.

It will be observed that the errors, due to the above-mentioned sources, would not have the effect of neutralizing each other, but that they would all tend toward reducing the velocity and the computed value of c, and that some of them, unless their absence has been fairly established, may have a very important, if not a vitiating effect upon the results.

The proper method of attachment of a piezometer has been fully discussed by the authors. One of the advantages of a multiplicity of connecting points with the main pipe is that the effect of any imperfection of workmanship, at any one point, is lessened. As it was deemed sufficient to have only a single point of attachment and as very small losses of head had to be measured, especial care became necessary to comply with the well-known requirements, that the hole be smooth and the channel leading away from it have a direction accurately perpendicular to that of the flow. It was stated that in connecting the manometer to the wooden pipe a hole was bored at right angles to the axis of the pipe, that a pipe was screwed into this hole, and that care was taken that there should be no projection beyond the inner surface. If, as may have been the case, this connection was made without first draining the pipe or relieving the pressure, it must have been a difficult matter to ascertain the true condition of the hole, and to insure the absence of ragged edges and projecting slivers; so that some further particulars on this point are desirable.

The plan of the pipe line under consideration (Fig. 9) shows a long curve at the upper end and a short curve further down, the aggregate length being about half that of the entire line. The profile, moreover, indicates a succession of vertical curves so that the aggregate length of the actual tangents, as compared to that of the curves and the greatest length of any one tangent is evidently quite small, especially if expressed

in diameters. In this respect conditions differ from those under which the earlier tabulated experiments were made. Whether the effect of such a succession of easy curves upon the grade line is at all appreciable must remain a matter of speculation until it shall have received some attention from experimenters. Practically nothing is known of the path described by individual particles of water, either in a straight or in a curved pipe. It is difficult to conceive that a motion parallel to the axis of the pipe, as assumed in theory, could be even approximately maintained in a pipe curving both horizontally and vertically and it is highly probable that the relative position of the particles in the successive cross-sections is constantly changing. Under favorable conditions of curvature and velocity, as yet entirely undetermined, this change may acquire a certain degree of regularity, causing the advancing body, as a whole, to acquire a rotating motion as has been observed in hydraulic mining, when the pipe leading to the nozzle is curved. It has been found that a rotating motion may at times become sufficiently pronounced to be troublesome, and cause the water to whirl as it comes out of the nozzle, thus interfering with the erosive effect of the stream. The difficulty is overcome by placing thin partition plates in the pipe a short distance back of the nozzle. This is mentioned as having possibly affected, to a slight extent, the frictional losses measured, and to show that this subject is worthy the attention of experimenters. It does, however, in no way account for the great difference between the anticipated values of c and those found by the authors. Mr. Henny.

CHARLES D. MARX, M. Am. Soc. C. E.; CHARLES B. WING, Assoc. M. Am. Soc. C. E., and LEANDER M. HOSKINS, Esq.—The foregoing discussion and correspondence deal partly with the question of the reliability of the experimental results given in the paper, and partly with other matters relating to the carrying capacity of large pipes. In the further discussion now offered, it seems desirable to keep such considerations as bear directly upon the question of reliability distinct from the other and more general questions that have been raised. Messrs. Marx, Wing and Hoskins.

RELIABILITY OF RESULTS.

In considering the degree of reliability of a series of experimental results such as that under discussion, three different kinds of evidence may be available. The first relates to the conditions under which the observations were made and the methods adopted, the second to the degree of accordance of the results among themselves, and the third to their agreement or disagreement with the results of other similar experiments. Of these classes of evidence the first two are always of fundamental importance, and especially is this true in a case like the present, in which the other data available for comparison are very meager. In preparing the paper, it was the aim of the

Messrs. Marx, Wing and Hoskins.

authors to describe fully and clearly the conditions and methods; to present the results in a form showing clearly their degree of accordance; and to present such data of similar experiments as seemed to be of direct value for purposes of comparison. In view of the questions that have been raised, some further consideration of these three phases of the subject seems desirable.

(1) *Conditions and Methods.*—Some of the questions raised relate to matters which are fully treated in the paper. In addition to what is there given regarding the conditions under which the experiments were made, the apparatus used, the methods of observation, and the method of reducing the results, the following may be said by way of direct reply to the questions raised in the discussion and correspondence:

It has been suggested that certain constants used in the work were not determined with sufficient precision. In particular, the specific gravity of the mercury used, the calibration of the gauges, and the diameter of the pipe have been called in question.

It is obvious that whatever error there may have been in the value of the specific gravity of the mercury affected only the value of the loss of head between piezometer stations. The authors felt justified in assuming that the value used (13.6) could be depended upon as correct to a degree greater than the possible degree of precision attainable in the observations. Table No. 10, given by Mr. Kuichling, shows that, so far as the effect of variations of temperature is concerned, the assumption is justified. It may be pointed out that, if the true value of the specific gravity is between 13.54 and 13.66, the error involved in the value 13.6 is less than $\frac{1}{2}$ of 1 per cent. An error of $\frac{1}{2}$ of 1% in the value of the lost head would affect f by the same relative amount, and c by half that relative amount, or $\frac{1}{4}$ of 1 per cent. Such an error is far within the possible limit of accuracy of the observations.

With regard to the calibration of the piezometers, it would appear that, since the position of the top of the mercury column was read directly by means of a fixed scale, the only calibration needed was the testing of the graduations of the scale and the determination of the height of the zero of the scale above the surface of the mercury in the reservoir. The degree of precision needed in the latter determination was not great, for reasons stated in the paper. The scales used were accurately graduated by a reliable maker, and were undoubtedly correct to an even greater degree of precision than the nature of the experiments necessitated.

The diameters of the steel and wooden pipes were not determined by direct measurement. The conditions made this impossible, even had it been desirable. The diameters were computed from the dimensions of the plates used (in case of the steel pipe) and of the staves

(in case of the wooden pipe). As the diameter of the wooden pipe especially has been questioned, it may be said that the value 72.5 ins. was taken from the data given in Mr. Goldmark's paper.* Messrs. Marx Wing and Hoskins.

"The pipe was built of thirty-two staves, the finished staves being $7\frac{1}{2}$ ins. wide on the outside, $7\frac{1}{8}$ ins. wide on the inside and 2 ins. in thickness. * * * Both sides of the staves were dressed in a planing machine to conform to the outlines of a circle; the outside to a circle of $38\frac{1}{4}$ ins. radius, and the inside to a circle of $36\frac{1}{4}$ ins. radius."

If, instead of taking the value here given for the inner radius, $36\frac{1}{4}$ ins., the radius is computed from the width of the staves, a slightly greater value results. The value of the diameter thus computed is 72.57 ins. or 72.69 ins., according to whether the width $7\frac{1}{8}$ ins. was measured along the circular arc or along the chord. It thus appears that the dimensions of the staves give a value of the diameter a little greater than the value used by the authors, rather than a little less, as stated by Mr. Henny. As regards the effect of the swelling of the wood upon the diameter, the authors can express no opinion, having no experimental knowledge. Neither can definite information be given as to changes in the shape and size of the cross-section due to earth pressure.

Two remarks seem pertinent in this connection: *First*, whatever uncertainty exists as to the value of the diameter of the stave pipe when in use, similar uncertainty will exist in case of any pipe line to be designed in the future. *Second*, the greatest possible error that can be attributed to this uncertainty is insignificant in comparison with the difference between the results shown by the Ogden experiments and those hitherto accepted by some engineers.

The question has been raised as to whether the observations may not have been vitiated by the presence of air in the water. There are two ways in which, conceivably, air may influence measurements of the kind under discussion—by accumulating at the summits of the pipe line, thus reducing the cross-section at these points, and by accumulating in the piezometer pipes. As to the former, it may be said that the air-valves along the entire pipe line were opened every morning for the escape of whatever air might have accumulated. Although it is true that air-valves are not present at all the summits of the wooden pipe, the slope of this pipe is everywhere so small that the accumulation of air seems improbable. Moreover, as remarked by Mr. Henny, the relief shaft at the tunnel would have the effect of freeing the water from air just before it entered the section of wooden pipe experimented on. It may be said further that if air in this pipe had been a serious disturbing factor, the results would have shown much greater irregularity than is indicated in Fig. 10. As to the possible effect of air in the piezometer connections, the authors fully recognized the necessity of keeping these connections free from air. As stated in the

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 268.

Messrs. Marx, Wing and Hoskins.

paper, an air-cock was in each case placed at the highest point of the pipe leading from the pressure-section to the piezometer, and this was frequently opened for the escape of air.

It may be said that, in the manipulation of the air-cocks, no noticeable amount of air was in any case found to have accumulated. That a small quantity did collect when the gauges were out of use for several hours was shown by the fact that slightly different gauge-readings were sometimes obtained before and after the cock was opened at the beginning of a series of observations.

The question as to the probability of sediment having been present in the pipe in quantities sufficient to affect appreciably its carrying capacity has been answered by Mr. Goldmark. It may be said further, that twice during the period covered by the observations the interior of the steel pipe was examined throughout its entire length, and that no important amount of sediment was found.

In reply to Mr. Noble's question regarding variations in the rate of discharge during the periods of observation, reference may be made to what is said under the heading "Plotting of Reduced Readings," page 480, and to the sample curves in Figs. 4 and 5. The steadiness of the rate of discharge is shown by the curves giving difference-gauge readings. At the time when the experiments were made, the consumption of water in the supply of power to customers was not subject to frequent variation, and the total rate of discharge could be adjusted, within certain limits, to the needs of the experiments.

The reason a longer section of the wooden pipe was not used in the experiments was on account of the existence of the open stand-pipe at the tunnel, permitting an overflow of water until the velocity reached about 1.5 ft. per second. It was desired to extend the observations to as low velocities as practicable, especially as, under the conditions then existing, the range was necessarily limited to velocities below 4 ft. per second.

In reply to Mr. Henny's question as to the attachment of piezometers to the wooden pipe, it is to be said that it was necessary to tap the pipe when under pressure, and that it was not practicable to insure perfect smoothness of the hole and freedom from splinters. That no serious splintering occurred was indicated by the smoothness of the issuing jet. The authors see no reason to believe that the presence of splinters at the inner edge of the hole would have an appreciable effect upon the indications of the piezometer.

Some remarks may be made by way of general comment upon the conditions under which the experiments were made, and upon the methods adopted. If comparison be made with similar experiments made elsewhere, it will be seen that the Ogden experiments were made under conditions some of which were favorable and others somewhat unfavorable.

The conditions for measuring the rate of discharge were exceptionally favorable, owing to the presence of the Venturi meters. The reliability of these measurements has not been questioned in the discussion. It is believed that, where Venturi meters are available, most engineers will prefer this method of measurement, both to the use of weirs and to direct measurement in a reservoir. While weir measurements of the most careful kind will doubtless yield results which many engineers would regard as fully equal in reliability to those obtained with Venturi meters, it is probable that this high standard has been reached in only a few of the weir measurements made in experiments on the carrying capacity of large pipes. Messrs. Marx, Wing and Hoskins.

As regards the measurement of lost head, where the pressure conditions permit the use of water piezometers, these are to be preferred to mercury gauges, since their indications are more precise in the ratio of the density of mercury to that of water. The experience of the authors leads them to believe, however, that if the mercury gauge is read to the thousandth of a foot, the last figure will usually be correct to within one unit. An error of 0.002 ft. would be equivalent to less than 0.03 ft. of water.

It is true that a great length of pipe is to be preferred to a small length, other conditions being equally favorable, since a given error in the measured loss of head is important in inverse ratio to the length. It may be suggested, however, that the use of the long pipe may not be without its disadvantages, among which may be mentioned the increased difficulty of avoiding the accumulation of air at summits.

It should be observed that the sacrifice of precision involved in the use of mercury gauges instead of water columns, and in the use of a comparatively short length of pipe, may be compensated by multiplying the number of observations. The observational errors in the values of the lost head are true "accidental" errors, as likely to be plus as minus. The importance of these errors, as affecting the mean of the series of results, is indicated by the degree of accordance of the results among themselves. This question will be discussed below.

The most unfavorable feature of the experiments was the leakage in the mercury gauge at the lower end of the steel pipe. This has been fully discussed in the paper, and needs no further comment here. The authors would only remark that, although the steel pipe observations show, on this account, less accordance than was expected, the general results are believed to be fairly reliable. Their reliability, in fact, has been accepted practically by those who have discussed the paper.

(2) *Accordance of Results.*—Some light is thrown upon the degree of precision of the results by a careful examination of their accordance among themselves.

It has been pointed out that the values of c for low velocities show very considerable discrepancies. The values of c do not, however,

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give direct indication of the degree of precision of the measurements. This is to be ascertained by comparison of the values of the quantities directly measured rather than of quantities computed from them. The values of the lost head, being obtained by taking differences of observed quantities, may be treated as if obtained by direct observation, and their accordance gives a direct indication of the precision of the observations; while the values of c, which vary inversely with r^2, magnify the errors very greatly for small values of v.

Referring first to the steel pipe observations, the best indication of the degree of accordance is afforded by Fig. 6. It is seen that the observations at low velocities show about the same accordance as the rest of the series.

If the curve shown in Fig. 6 be taken as representing the mean values resulting from the series of observations, the probable error of an observation may be estimated by computing residuals from the mean values, and applying the usual formula. These residuals are found by subtracting from each "observed" value of H' the value given by the curve for the same velocity. Applying the least-squares formula, the probable error of a single observation is found to be 0.0142. This is the probable error of an observed value of the loss of head per thousand feet. Multiplying by the length of the pipe expressed in thousands of feet, we find 0.062 ft. as the probable error of an observed value of the total loss of head.

The error in c caused by a given error in H' may be found by differentiating the formula $v = c\sqrt{rs}$, which may be written

$$c = 25.8\,\frac{r}{\sqrt{H'}}.$$

If δc is the error in c caused by an error $\delta H'$ in H', we have

$$\delta c = -\frac{12.9\ r}{\sqrt{H'^3}}\,\delta H'.$$

Using this relation, and assuming 0.0142 as the probable error of H', the values of the probable error of c for different values of v are as follows:

v	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
P. E. of c	59	15	6.5	3.7	2.3	1.6	1.2	0.91

It should be noticed that these values denote the probable error of the value of c derived from a single observation, not that of the adopted mean values. Of the degree of precision of these mean values, no very definite estimate can be made, since the values of H' given by the curve in Fig. 6 for different values of v are not obtained by averaging several direct observations on a quantity whose true value is constant, and no

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accurate estimate can be made of the number of observations contributing to the average result adopted for a given value of v. The mean values must, however, be regarded as having a decidedly higher degree of precision than a single observed value.

The sinuous character of the mean curve showing values of c (Fig. 7) has been referred to as casting suspicion on the results. As stated in the paper, the authors do not regard the experiments on the steel pipe as sufficiently precise to exclude the possibility that this sinuosity may be due to uncompensated errors. It may be remarked, however, that such a variation of c with the velocity cannot be regarded as extremely improbable *a priori*, especially if c is nearly constant, as seems to be the case for considerable ranges of velocity in large riveted pipes. A constant value of c would mean that the loss of head varies as the square of the velocity. This law would give, in Fig. 6, a parabola with vertex at the origin and axis vertical. It is entirely possible to assume a curve intersecting a given parabola in several points, without implying any irregularity in the law connecting H' and v. The curve shown in Fig. 6 implies a fairly regular law, notwithstanding that the corresponding curve for c is sinuous.

If the precision of the observed values of H' for the wooden pipe be estimated in the above manner, adopting the mean curve shown in Fig. 10, the results reached are the following:

Probable error of $H' = 0.0123$.

Probable error of an observed value of the total loss of head between pressure stations $= 0.0123 \times 2.71 = 0.033$.

If the very discrepant observation No. 46 be omitted, these values are reduced to 0.00802 and 0.022. Adopting the larger values, the probable error of the value of c obtained from a single observation, computed as in the case of the steel pipe observations, has the following values for different velocities:

v.....	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
P. E. of c.....	21	9.5	5.4	3.5	2.5	1.9	1.4	1.1

Referring to Mr. Noble's statement that the experiments appear to show no uniformity in the relation between mean velocity of flow and loss of head, it may be pointed out that, in the case of the experiments on the wooden pipe this relation appears to be well represented by the equation

$$H' = 0.0314\, v + 0.0343\, v^2.$$

The fact that this relation holds, within the range of the experiments, does not, of course, warrant its application to higher velocities.

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(3) Comparison with Other Experimental Data.—Many data relating to the carrying capacity of large riveted pipes are available. Although a considerable gap exists between the size of the Ogden pipe and the sizes of the largest pipes previously experimented on, the results appear to be accepted as in agreement with what previous knowledge would lead us to expect.

In the case of wooden pipe very little experimental knowledge is available for comparison. Reference is made in the paper to three experiments, and to these Mr. Adams has now added a fourth. The published accounts of the gaugings of the Denver and Butte City conduits are very meager, and without further information as to the methods used, the results do not appear to be entitled to great weight.* The diameters of these conduits are 30 ins. and 24 ins., respectively, while those of the pipes experimented upon by Mr. Adams at Astoria and at Los Angeles are 18 ins. and 14 ins., respectively. Experiments with pipes of these sizes, however reliable, do not afford a safe basis for estimating the capacity of a conduit 72 ins. in diameter.

It is true that in the absence of experimental knowledge obtained directly from wooden pipes, many engineers have felt justified in using the results of experiments made upon smooth cast-iron pipes, and upon open conduits with comparatively smooth surfaces. Even on this basis not many data are available for estimating the capacity of pipes as large as 6 ft. in diameter. So far as known to the authors, the experiments of Stearns and FitzGerald are the only careful gaugings made of smooth pipes of diameters as great as 4 ft., previous to those at Ogden.†

The curves given by Hamilton Smith, Jr., M. Am. Soc. C. E., showing the relation between c and v for different diameters up to 8 ft., appear to be, for the larger sizes, mainly speculative, obtained by assuming that the laws observed to hold for small pipes may be expected to hold for large sizes also. Such an extension of a purely empirical law far beyond the limits of the data upon which it is based can afford no reliable basis for estimating the credibility of a new set of experimental results.

To reason from cast-iron or cement-lined pipes to wooden pipes involves the assumption of equal "smoothness." To reason from

* Mr. D. C. Henny informs the authors that the Butte City gauging was made by him, and that the method used in measuring the velocity of flow was not such as to insure a high degree of accuracy. In regard to the Denver experiment, the statement of Mr. Schuyler, in the paper already cited, that the discharge measurements were checked by the use of a current meter, indicates that the standard of accuracy was not high.

† In the translation of the work of Ganguillet and Kutter, by Hering and Trautwine, the result of an experiment, by Jas. M. Gale, on a new cast-iron pipe 4 ft. in diameter, and $3\frac{3}{4}$ miles long, is given. The value of c is 112.4 for a velocity of 3.458 ft. per second, the corresponding value of n being 0.0168. It will be noticed that this value of c is very decidedly lower than those found by Stearns and FitzGerald. The method of measurement is not stated, and the degree of reliability of the results is unknown.

open conduits of various shapes to a closed circular pipe involves the assumption that the effect of the form of the cross-section upon the loss of head is wholly accounted for in the value of the hydraulic radius. In the opinion of the authors neither assumption can be justified theoretically, or (in the present state of knowledge) experimentally. The words "rough" and "smooth," when applied to the inner surface of a conduit, must be understood as referring to the total effect of the inner surface in retarding the flow. Whether the adhesion of water is greater or less for a surface of water-soaked wood than for an asphaltum-coated surface of cast iron can be known only by experiment. Messrs. Marx Wing and Hoskins.

Causes of Loss of Head.

Among the questions considered in the correspondence is that of the causes of the losses of head observed in the steel and wooden pipes. No attempt was made by the authors to estimate, separately, the effects of the various conditions which obviously contributed to the total losses observed. The values found for H', c and f are to be regarded as resultant values, due to the combined effect of all resistances.

Mr. Kuichling's method of estimating the loss of head due to rivet heads or other protuberances is to be justified, if at all, by comparison with experimental results, not only in this case, but in many cases. The Weisbach formula, upon which the method is based, must be regarded as empirical, and this application appears to be a very radical extension of the limits within which the formula is to be used.

Mr. Le Conte suggests that the value of c for the steel pipe is a function of the thickness of the plates of which the pipe is made. In the case of lap-jointed pipes, whether "taper" or "cylinder," it is reasonable to suppose that the thickness of the plates is one of the factors governing the loss of head, but it is not evident that this is the case when butt joints are used, as in the Ogden pipe.

It may reasonably be assumed that a portion of the loss of head, in both the steel and the wood pipes, is due to bends. The amount of this effect is a matter of great interest and importance; but in the present state of experimental knowledge it seems useless to attempt a quantitative estimate. The authors do not regard it as improbable that a straight pipe, otherwise similar to the wooden pipe experimented upon, would give values of c materially higher than those found. As a guide to future design, however, the results do not lose value by reason of the bends, since in practice the existence of bends will be the rule rather than the exception.

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Applicability of Kutter's Formula.

In the opinion of the authors, no rational formula for the discharge of pipes is in existence, and the complexity of the conditions affecting loss of head is such as to offer little ground for the belief that such a formula will be discovered in the future. So far as theoretical basis is concerned, the Chezy formula is as satisfactory as any.

The Kutter formula is merely an attempt to express the value of c in the Chezy formula as a function of three quantities, "roughness" of surface, hydraulic radius, and hydraulic slope. It is purely empirical, and is based wholly upon experimental data obtained from open channels. The authors of the formula designed it for application only to open channels, and although it is now quite extensively applied to closed pipes, the data upon which this extension is based include few experiments upon large circular conduits. The authors have pointed out in the paper that no constant value of n (the coefficient of roughness) will bring their results, for either the steel or the wooden pipe, into even approximate conformity with the formula. The variation of n with the velocity is shown in the following table:

v.	n.	
	Steel pipe.	Wood pipe.
1	0.0135	0.015
2	0.0142	0.0137
3	0.0144	0.0131
4	0.0145	0.0128

The opinion was also expressed that the different values of n shown in Table No. 4 for new riveted pipes of various sizes could not be attributed to differences in roughness. Further, it was pointed out that the experiments of Mr. Adams at Astoria, on an 18-in. stave pipe, when compared with those made at Ogden by the authors on 72-in. pipe, show that the value of n for stave pipe must be regarded as varying with the diameter, if Kutter's formula is to be applied. The variation of n with the velocity has been noticed in various other cases, and is mentioned by several writers. In view of these facts, the opinion expressed in the paper, that Kutter's formula must be regarded as defective so far as applicability to large pipes is concerned, appears to be fully justified.

MEMOIRS OF DECEASED MEMBERS.

WILLIAM EZRA WORTHEN, Past-President and Hon. M. Am. Soc. C. E.*

DIED APRIL 2D, 1897.

William Ezra Worthen, Past-President of the American Society of Civil Engineers, died at his residence, in New York City, on April 2d, 1897, after an illness of some months.

Mr. Worthen was born at Amesbury, Mass., March 14th, 1819; his father, Ezra Worthen, having been one of the projectors of Lowell as a center of manufactures, and the first Superintendent of the Merrimac Mills from date of June, 1822. William E. Worthen, after having been fitted for college by I. B. Hayward, of Boston, was entered at Harvard, and graduated from that institution in 1838.

He commenced the profession of civil engineering under the tutorship of S. M. Felton, an assistant in the office of the then prominent engineer, Colonel Loami Baldwin; and the day after graduation from Harvard was put to work under the direction of George R. Baldwin, measuring the flow of the water used at the Merrimac Mills. With Mr. Baldwin he went to Boston and was employed in the surveys and brook measurements for an increased supply for the Jamaica Ponds Water-Works, a private enterprise for supplying Boston with water by gravity. He then returned to Lowell, and, under the late James B. Francis, Past-President and Hon. M. Am. Soc. C. E., was engaged for some time in general hydraulic and mill work.

In 1840 Mr. Worthen was with Mr. G. W. Whistler on the Albany and Stockbridge Railroad, commencing with the preliminary surveys of the road and remaining until its completion; 7 miles of the construction work having been under his immediate charge. Returning to Lowell he was again engaged with Mr. Francis in hydraulic work and the construction of the lower end of the Northern Canal; he then designed and built a dam and the mills on the Suncook, at Suncook, N. H. He also designed and constructed a dam and mills for the Boston Manufacturing Company, on the Charles River at Waltham, and the Suffolk, Tremont, Lawrence, Appleton and Hamilton Mills at Lowell. He reported on the water supply of Lowell, and for a time was in charge of the cotton mills and machine shops of the Boston Manufacturing Company, as Acting Superintendent.

* Memoir prepared by Julius W. Adams, George S. Greene, Henry Flad, Joseph P. Davis and Alphonse Fteley, Members, Am. Soc. C. E.

After a visit to Europe, in 1849, Mr. Worthen came to New York City and to some extent devoted himself to architectural work, building the structures at No. 200 Broadway. At the same time he took part in editing several mechanical publications. In 1851 he was in charge of the cotton mills and machine shops of the Matteawan Company at Fishkill; but, returning to New York in the following year, he was again engaged as an architect, designing and constructing, among other buildings, the bindery of the Appleton Publishing Company, in Franklin Street, and their present works in Williamsburg. Later, he was Engineer of the New York and New Haven Railroad and Vice-President of that road until 1854, under Mr. Robert L. Schuyler.

He designed and built the dam across the Bronx River at West Farms; and then, opening an office in New York City, supplemented his architectural and engineering practice by constructive iron-work and steam heating for buildings. He was engaged for a long time in general engineering work; building the dam across the Mohawk at Cohoes, N. Y., testing steam pumping machinery at many points, and designing pumping engines for Mr. Kirkwood for his reports on the water supply of Cincinnati, and of St. Louis. He devised floating grain docks for the same engineer at the Jersey City depots of the Erie Railroad Company, and was Sanitary Engineer of the Metropolitan Board of Health, of New York City, during its whole existence, 1866–1869.

Mr. Worthen had much practice in the measurement of the flow of water in canals, reporting upon this class of work for Paterson, Trenton, Passaic and Indianapolis, among the more important. He finished the Southern Boulevard in New York City; built the first pumping engine at High Bridge, New York City; the engine at Rockville Center and many pumping engines of smaller capacity; extended the water supply of the City of Cohoes and finished the Water-Works of Long Island City.

As Consulting Engineer he, at various times, examined and reported upon the Indianapolis and the Hartford Water-Works, the condition of the docks of New York City, the Riverside Park, the extension of the streets of Brooklyn, the selection of pumping engines for Boston, also the testing of large pumping engines at Brooklyn, Lawrence, Jersey City, Philadelphia, St. Louis, Milwaukee and other points. For this class of work his services were in very general demand for many years. In 1890 and 1891 he served as Chief Engineer of the Chicago Main Drainage Canal. He was Engineer of the later Commission on Rapid Transit in the annexed district of the City of New York, and reported upon the water-power of the Falls of St. Anthony, Minn.

He served on a committee of the Society appointed to investigate the failure of a dam at Mill River, Williamsburg, Mass., his associates

being James B. Francis and Theodore G. Ellis. Their report will be found in *Transactions*, Vol. III, page 118.

Mr. Worthen was a frequent contributor to the professional discussions of the Society, and presented the following papers: "Improvement of Sedimentary Rivers;" *Transactions*, Vol. XX, page 230; "Concrete Sewers at Mount Vernon, N. Y.," Vol. XXIV, page 393; "Steam Heating," Vol. XXIV, page 206.

This very incomplete sketch will give some idea of the varied practice which had occupied Mr. Worthen during a long and most active professional life, there being scarcely a branch of civil or mechanical engineering wherein his professional fitness has not been conspicuous in a marked degree. It was not his lot to project or carry to completion great works of internal improvement, such as challenge the admiration of the unthinking public, but in a very unobtrusive way he continually rendered that essential service toward the furthering of enterprise, without which the best conceived projects would prove abortive. To a remarkable power of rapid generalization, seemingly incompatable with painstaking accuracy, he united an almost intuitive perception of the requisite expedients of detail and design. His quickness in technical analysis, combined with the before-mentioned qualities, rendered possible the successful completion of many important works with which, owing to a forgetfulness of self, his name is scarcely associated. A retentive memory, to sift and treasure the facts in science and art which extended study had opened up to him, the tact of judicious selection and application, originality and boldness at times bordering on audacity, and a positiveness that silenced all opposition, have been the characteristic features of his long practice.

Whatever Mr. Worthen may have owed to study or experience in the arts of construction, his acknowledged genius (using the word in its strictest sense) stamped him as an engineer, *nascitur non fit.*

Mr. Worthen possessed an overflowing vein of wit and humor; which served to temper the asperities not unusual in professional debates; this, coupled with a kindliness of disposition which could see nothing in others, to speak of, but what was commendable, led to his friends being numbered only by his acquaintance. His social relations were of the happiest kind, and his memory will long be prized by all who knew him.

He married Miss Margaret Hobbs, of Boston, who survives him.

Mr. Worthen became a Member of the American Society of Civil Engineers December 4th, 1867. He was elected President of the Society January 19th, 1887, and served one term; he was made an Honorary Member of the Society April 4th, 1893.

BURR KELLOGG FIELD, M. Am. Soc. C. E.*

DIED JANUARY 13TH, 1898.

Burr Kellogg Field was born at Auburn, Ind., on the 5th day of May, 1856. Soon after his birth his parents moved to Malden-on-the Hudson, in New York State. He prepared for college at the St. John's Military School at Sing Sing, N. Y., and entered the Sheffield Scientific School of Yale University in the fall of 1874, graduating with the Class of 1877. His college career was not marked with any extraordinary incidents except his great popularity among his associates. Owing to his exceedingly affable, pleasant and generous ways he was christened by his associates "The Cherub," by which name he was known among his intimates until the last day of his life. Mr. Field's parents were wealthy, and he was denied nothing during his college course which would add to his happiness or contentment or in any way assist him in the prosecution of his studies. Ten days before commencement his father failed in business, and Mr. Field graduated from college in a worse than penniless condition, being obliged to borrow money from his associates to pay his ordinary student's bills in and about New Haven.

Burr K. Field commenced his engineering career after graduating from the course of civil engineering at Yale University as a water boy to a section gang on the Baltimore and Ohio Railroad at the salary of $3 per week. Every engineer knows of the struggle for existence of the young engineering graduates during the period of 1877, 1878 and 1879, and Mr. Field's case was no different, except that circumstances forced him to provide for the support of his aged father and mother and younger brother and sister out of his scant earnings.

In July, 1878, he entered the service of the St. Louis and San Francisco Railroad, in the department of tracks, bridges and buildings, where he remained until September 10th when he entered the employ of the Denver and Rio Grande Railroad as a rodman on the New Mexico and Colorado Division. After leaving the Denver and Rio Grande Railroad he was engaged as a rodman on the construction of the Omaha extension of the Old North Missouri Railroad until August, 1879, when he was appointed as a rodman on the Wichita extension of the St. Louis and San Francisco Railroad, in which place he remained until the spring of 1880, occupying successively the positions of rodman, leveler and topographer.

In the spring of 1880 J. F. Hinckley, M. Am. Soc. C. E., was detailed by the St. Louis and San Francisco Railway Company to

* Memoir prepared by Charles M. Jarvis, M. Am. Soc. C. E.

make surveys from what is now Monett, Mo., to Van Buren, Ark., and Mr. Field was engaged by him as topographer. He remained with Mr. Hinckley until the completion of the preliminary surveys. Much of the country traversed was wild and broken in the extreme, and little or nothing was known about it previous to these surveys. Mr. Field continued as topographer on location, under A. P. Man, M. Am. Soc. C. E., principal assistant engineer, and later under Mr. M. N. Randall, locating engineer. The work covered the territory between Fayetteville, Ark., and Van Buren, Ark., the line crossing the Boston Mountains, near where the present station of Winslow is located. Later Mr. Field was placed as engineer in charge of masonry, trestles and bridging on the line between Winslow and Van Buren. The masonry and bridging on this portion of the line is important in character, the streams crossed being rapid torrents, and the foundations being difficult to sink. Under his direction there were constructed three iron trestles ranging from 400 to 700 ft. in length and from 100 to 120 ft. in height, besides some twenty spans of Howe truss bridges resting on masonry piers. Upon the completion of this work in the summer of 1881, Mr. Field joined one of the surveying parties under the charge of R. L. Van Sant, M. Am. Soc. C. E., to complete the location between Van Buren and Fort Smith, and to make preliminary surveys through the country from Fort Smith to Paris, Tex. He remained in this position until February, 1882, when he was appointed Assistant Engineer on the Northern Pacific Railroad.

James Dun, M. Am. Soc. C. E., now chief engineer of the Atchison, Topeka and Santa Fé Railroad, who was at this time chief engineer of the St. Louis and San Francisco Railroad, speaks of Mr. Field in the following terms:

"My recollections of Mr. Field are of the most favorable nature, and I shall always retain the highest impressions of his character, professionally and personally. During the time he was engaged upon the St. Louis and San Francisco Railroad he was associated with men more than ordinarily bright, and who have since all won an honorable place in the profession which they adopted; and among them all I know of no one who promised to attain more than he. He was universally liked by his associates and by all who came in contact with him, and his untimely taking off was deeply regretted by all of his earlier friends and associates in the West."

From February, 1882, to November, 1883, Mr. Field was Assistant Engineer on the Northern Pacific Railroad, part of the time in charge of the construction of the Yellowstone Division, and later in charge of the tracks and bridges in the construction of the 52 miles of the National Park Branch, connecting the Yellowstone Park with the Northern Pacific.

On January 17th, 1884, he was appointed by Mr. John D. Estabrook, then Chief Highway Commissioner of the City of Philadelphia,

to the important position of Superintendent of Bridges in the Highway Department of the City of Philadelphia, "which position he filled in a thorough and conscientious manner, ever looking out for the public welfare regardless of party pressure or selfish ends."

Two years later in 1886 Mr. Field accepted an appointment as Assistant Engineer of the Berlin Iron Bridge Company, of East Berlin, Conn. His advancement with this company was very rapid, and at the time of his death he occupied the important position of Vice-President, having full charge of the making of all contracts. During Mr. Field's connection with this company its business was much extended, and its product introduced into all parts of the world. In achieving this Mr. Field had no small part, and his death has been a severe loss, not only to the company, but also to his associates.

Mr. Field was an indefatigable worker, not only for the corporations which he so faithfully served, but in every walk of life. He was a friend of his fellow-men, a devoted and earnest worker in the church, a staunch friend to the cause of temperance, and active and energetic in every public position that engaged his attention. To quote from the words of a friend:

"There is a vacant place—yes, not one but many vacant places in our midst, and his absence from these we know and feel is our great loss. There are vacant posts of duty which were always well filled. How many and varied they were! The posts of great business responsibility; of influence in Church work; of prominence in the efforts for good citizenship. The earnest and conscientious life which occupied these places has been taken from us, and we feel our loss. But not only in the large and more important positions do we know that a loss has come, but such was the largeness of his life and efforts that within the small places there is a vacancy; there are vacant places in those higher realms of life's experience—friendship has lost a true friend; earnest and conscientious service in the Church has lost a bright example; Christianity has lost a good Christian. Think of what is good and bright—he stood for that. Think of what is true and earnest—he stood for that. Think of what is unselfish and noble—he stood for that. Think of what is tender and sympathetic—he exemplified that. From whatever standpoint we view his life, as a business man, a citizen, a member of the Church, a friend, or the head of a family, we behold one who stood for what was best and true in all these. To say such of Burr K. Field is not to speak flattery or extravagant eulogy, but only to speak the sober truth."

In the flush of manhood he was taken away without an instant's warning.

While engaged with the St. Louis and San Francisco Railroad, he became acquainted with Miss Juanite Bourland, the daughter of Doctor Bourland, of Van Buren, Ark., to whom he was wedded on the 5th day of May, 1886, and who, with two children, survives him.

Mr. Field was elected a Member of the American Society of Civil Engineers on October 1st, 1884.

THOMAS DAVIS LOVETT, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1897.

Thomas Davis Lovett, son of Captain Pyam Lovett and Lucy Davis Lovett, was born June 10th, 1823, in Beverly, Mass., where he grew to young manhood, receiving his education from the Beverly Academy, and later in New York City. He was descended, both on the paternal and maternal side, from honorable ancestors, and was a great grandson of Benjamin Lovett, one of the one hundred and five minute men who marched from Beverly to Concord, October 19th, 1775. While at the academy his father had him employ the usual Wednesday and Saturday half holidays in learning the routine of book-keeping and commercial transactions as they occurred in the Beverly Bank, of which Captain Lovett had for many years been president. It was the father's expectation that the son would succeed him in that office of trust and responsibility, but after his return from school at New York he began the practical study of civil engineering under Frank Edwards, at Beverly, about 1841 or 1842.

With that principal, his first railroad employment was in the construction of what is now the Gloucester Branch of the Boston and Maine Railroad, and next in the construction of the road then known as the Portland, Saco and Portsmouth Railroad, now also a part of the Boston and Maine system. Soon afterward he was employed on the Troy and Boston Railroad, having, for a time, at least, responsible charge of maintenance of certain portions constructed, this leading at a later period to his rebuilding the bridge masonry on the branch to historical Bennington.

The surveys for the eastern portion of the Hoosac Tunnel Line, known as the Troy and Greenfield Railroad, incorporated under the auspices of the Commonwealth of Massachusetts, followed in 1848 and 1849, or later. A part, at least, of these surveys, and possibly those for the entire line, were made by Mr. Lovett. He and a young English draughtsman, a Mr. Dillon, prepared extensive and elaborate maps of the line, including the great tunnel, which were, unfortunately, all destroyed by fire some years after. While this and the corresponding work was under way, he moved from Greenfield, Mass., to Shelburne Falls, thence to North Adams, and thence, upon the completion of these undertakings, to Troy, N. Y., where he built the first street-car line of that city.

In 1853, Mr. Lovett moved to Chillicothe, Ohio, with Captain Archibald Kennedy (formerly of the Vermont Central Railroad), and,

* Memoir prepared by W. B. Ruggles and Samuel Whinery, Members, Am. Soc. C. E.

associated with John Waddle, M. J. Becker and Jacob Blickensderfer, all of whom, in later years, were prominent in railroad and State improvements, was soon employed in the location and construction of the Marietta and Cincinnati Railroad. Mr. Lovett, as assistant engineer, had charge of part of the construction from Chillicothe, west, toward Blanchester, and of surveys from Blanchester to Cincinnati. The lines of these surveys were direct and permitted of lighter grades than those finally adopted; but, as much of the projected location was in close proximity to the Little Miami Railroad, that road was for many years used as an entrance into the city, and the final location to Cincinnati, finished in 1869, was in a great measure determined by the necessity of reaching connecting lines in Cincinnati.

While making these surveys he lived at Goshen, Ohio, but had an office on Third street, Cincinnati, not far from that occupied by him during the last years of his life, nearly forty-five years later.

In 1855, upon the completion of his work on the Marietta and Cincinnati Railroad, Mr. Lovett went to McArthur, Ohio, on the line of that road, and associated himself with others for the purpose of developing the coal fields of the vicinity, and, incidentally, he laid out and built up the town of Zaleski. Later, he removed to Chillicothe, but continued his connection with the mines until they were sold to English parties. While these enterprises did not prove financially very profitable to Mr. Lovett, he there acquired the experience and practical knowledge of coal operations that were subsequently valuable on the Ohio and Mississippi Railway and the Cincinnati Southern Railway.

Immediately after the sale of these mines, and in February, 1860, he removed to Cincinnati, were he was for a brief time employed in the City Water-Works Department. His early experience on the city passenger tramways of Troy was soon the means of his engagement for the construction of the Third Street line in Cincinnati, the first street railroad built in that city, and, shortly afterward, of the John Street line, the next road completed.

It was at a time when the popular prejudices of the city were against these, so far locally untried, public conveniences, and it required, not only his ability as a constructing engineer, but much diplomatic and skilful management to carry the project to successful completion.

Following these experiences, and in the same year (1860), Mr. Lovett entered the service of the Eastern Division of the Ohio and Mississippi Railroad Company as Chief Engineer, and continued with that road, with one interval of other service, until 1872. At the time of his appointment, George B. McClellan, the famous general of a few years later, was General Superintendent of the road, and placed implicit confidence in Mr. Lovett, giving him complete charge of the

engineering of the road, to the extent of not even giving any direct or specific instructions in so grave an emergency as the collapse of a bridge.

Mr. Wm. F. Biddle, who was at that time Division Superintendent, has, in a late memorandum, recorded the thoroughness with which Mr. Lovett looked after the details of his work, and says : "I have never forgotten the appreciation and admiration that were inspired by Lovett's ability, capacity and devotion to duty during that very trying time."

During Mr. Lovett's first term of service with the road many improvements were made, and many more wisely projected, but the emergencies of the war and the delay incident to the liquidation and reorganization of the company were such that the important measures urged by Mr. Lovett could not be carried out, but time has developed their wisdom.

Mr. Lovett left the service of the road, and, in 1864, went to Boston, and there acted as engineer for the Messrs. Stanton, who were engaged in general contracting work.

He maintained, however, an active interest in the affairs of the Eastern and Western Divisions of the Ohio and Mississippi Railroad Companies, which were, in 1867, reorganized as one corporation. The immediate occasion of his return to the West was the failure of the bridge over the Great Miami River. Mr. W. D. Griswold was, at the time, president, and he at once telegraphed Mr. Lovett to return; this he did on November 1st, 1867, and for seven months gave unremitting attention to the reconstruction of the masonry and superstructure of this bridge.

Then followed a period of active reconstruction and improvement on the road, in the course of which all the principal bridges were renewed in iron, the heaviest grades reduced, coal fields developed, and the terminal facilities of East St. Louis and Cincinnati enlarged, making such provision for the future business of the road that no radical changes and but few extensions or rearrangements have since been found necessary.

The branch line from North Vernon to Louisville was built under Mr. Lovett's direction and was completed in 1871. This is a monument that, even without his later and more extensive undertakings, would have established his reputation for judgment and ability; for it is seldom, in a region where 1% grades are considered very favorable, that a cross-country direct line 50 odd miles long can be so located as to require only 0.8% grades, and these falling, with two short exceptions, in one direction, making practically a continuous grade from the high ground at North Vernon to the Ohio River.

There still remained to be made one important change to put the road on a basis which would maintain its supremacy. The Ohio and

Mississippi Railroad, constructed by engineers and contractors coming from the Erie Railway, was laid with the corresponding 6-ft. gauge, and formed, with that road and the Atlantic and Great Western, a continuous broad-gauge line from New York to St. Louis. It was Mr. Lovett's final duty to change to standard gauge of 4 ft. 9 ins. the western section of these systems.

The Ohio and Mississippi was the first railroad of any considerable length to attempt the change of gauge of the whole line without any serious interruption of the train service, and Mr. Lovett was a pioneer in this as in many of his other undertakings. The change from 6 ft. to 4 ft. 9-in. gauge was so considerable that it was necessary to move both rails.

The most complete and careful preparations were made, and all the minute details were worked out and provided for by Mr. Lovett. There still exist among the papers of the company the full and complete printed instructions and tables prepared by him for the work. On Sunday, July 16th, 1871, the change of gauge was made on the Louisville branch and furnished a preliminary test of the plans adopted. One week later, on Sunday, July 23d, 1871, the gauge of the main line was changed from Cincinnati to St. Louis, a distance of 335 miles, in the remarkably short time of seven hours. It was at the time justly considered a remarkable feat, and its successful accomplishment illustrated Mr. Lovett's great executive ability. The experience gained in this work rendered Mr. Lovett's assistance valuable to other roads whose gauge was changed later, and he was called in consultation in a number of cases.

Late in 1872 his connection with the Ohio and Mississippi Railway Company ended, and in October of that year he entered the service of the City of Cincinnati as Principal and Consulting Engineer for the construction of the Cincinnati Southern Railway.

Surveys for this long-contemplated thoroughfare to the South had been made under the direction of W. A. Gunn, and the necessary State and municipal legislation secured and appropriations provided. Of these experimental lines, some were to Nashville and some to Knoxville and many to Chattanooga, and the first matter to be determined, after Mr. Lovett took hold of the project, was the selection of a line for final location. Each of the routes had its earnest advocates, the choice lying principally between that to Knoxville and those to Chattanooga. This problem was solved, by the adoption of one of the lines to Chattanooga, and the work of construction was soon under way, the first contract for grading having been signed in December, 1873. The magnitude of the undertaking is indicated less by the total length of the new line, 336 miles, than by the character of that line. To provide adequately for the construction of nearly 5 miles of tunnels at twenty-seven points, fifty-seven bridges, aggregating more than 2 miles in

length and 3 miles of viaducts, might, in a large part, be delegated to subordinates, as becomes the discreet and judicious manager of large affairs in these busy times; but Mr. Lovett, in addition to the usual general supervision, looked closely into the details of all this work. Not only was the construction of the Cincinnati Southern Railroad one of the largest and most important public works of the period, but it must be remembered that some of the structures were at the time unique in engineering experience and practice.

High Bridge, over the Kentucky River, for instance, was then not only the highest bridge on the continent, and the highest railroad bridge in the world, but it was probably also the first bridge built on the cantilever principle without falsework. While the direct credit for its design and construction belongs to the late C. Shaler Smith, the opportunity might never have been afforded to him had not the chief engineer of the road been a man of broad views and bold engineering conceptions.

The channel span of the bridge over the Ohio River at Cincinnati was, at the time it was built, decidedly the longest trussed girder span in the world, and exceeded what at that time was considered the extreme practicable limit of iron truss spans. Its history furnishes another illustration of Mr. Lovett's professional courage.

The Ohio being a navigable stream, the location for the proposed bridge was referred to the usual Board of United States Engineers. The law required an opening of 400 ft., but a controversy arising as to whether the crossing should be at Smith Street, as selected by the Board, or at Ludlow, as desired by the representatives of the river interests, the Board gave, as an ultimatum, the alternative of the required 400-ft. opening at Smith Street or of a 500-ft. opening at Ludlow, stating in effect at the same time that they considered a span of that length impracticable. Mr. Lovett, however, at once chose the Ludlow site, and the span was made 515 ft.

Considering that in addition to heavy and costly work on other portions of the line, that part of the road from the vicinity of the Cumberland River to the Emory River, crossing the Cumberland Plateau, is the heaviest and most expensive continuous stretch of railroad of equal length in America, some idea may be formed of the magnitude of the enterprise and the difficulties that had to be successfully met.

In December, 1876, when nearly the whole of the work had been let and the greater part of it completed, Mr. Lovett tendered his resignation as principal and consulting engineer, and soon after went abroad, residing for some time in Paris where there was at the time of the Exposition of 1878 an Exhibit of the Cincinnati Southern Railroad. This display led to his being engaged by French capitalists to make surveys for a line of railroad from Jerusalem to Joppa. It is to be regretted that little is known of his experiences there, further than that

he endeavored to have all points of historical interest, as far as possible, rendered accessible; but, that a practicable line was developed, is shown by its later construction.

Mr. Lovett, however, found the work very trying, physically, and one effect, little suspected by even those who knew him well, was the loss of the use of one eye. Sick with a prevailing fever, he returned to the United States, and soon after opened, on Third Street, in Cincinnati, an office as Consulting Engineer.

While connected with the Cincinnati Southern Railway Mr. Lovett had become interested in the possibilities of the coal fields along the road, and, shortly after his return, invested in the "Beaver Creek and Cumberland River Coal and Railroad Company," which investments he held until his death. He was actively engaged during the succeeding years in the management and development of this property, and of his real estate at Beverly, Mass., and in Kansas.

In the intervals of professional engagement, he, in one way or another, had been connected with many other enterprises. While acting as Chief Engineer of the Ohio and Mississippi Railway, he was made engineer for the Front Street Connecting Railway in Cincinnati, and in that capacity designed the combination rail used in the track. He was Consulting Engineer for the first of the many inclines which were a distinguishing feature of Cincinnati suburban development. He had a remote connection with the erection of the substructure of the Sabula Bridge of the Chicago, Milwaukee and St. Paul Railroad over the Mississippi River, and an active interest in the erection of the substructure for the St. Charles Bridge over the Missouri River. He traveled much, both professionally and for recreation, and in the course of a busy life had been in every quarter of the globe. The attraction of life increased with his years, and to the last he participated in the many gatherings of professional men, where he was always a welcome guest.

Mr. Lovett selected his subordinates with rare judgment. On the Ohio and Mississippi and the Cincinnati Southern Railways he had, as his assistants, T. M. R. Talcott, R. L. Read, Isaac Smith, R. L. Engle, G. Bouscaren, Samuel Whinery, H. R. Weeks, George B. Nicholson, W. P. Clark, Cabell Breckinridge, Robt. B. Stanton and Charles L. Strobel, all of whom have since been prominent as engineers and nearly all of them members of the American Society of Civil Engineers, and active in its affairs. It is doubtful if a better corps of engineers than that on the Cincinnati Southern Railway was ever organized.

Mr. Lovett was married September 8th, 1846, to Sarah Frances Folger, of Nantucket, who survives him. Of four children, two are living, Mrs. G. W. Pohlman and Mr. Pyam Lovett. He was a man of strong and positive character, great energy and of superior executive

ability; quick to decide and forceful to execute; bold and courageous in his opinions and frank and plain spoken in expressing them; fair and honorable in his business relations and loyal to his friends. Intolerant of deceit and trifling, he was a just and considerate superior, reposing confidence in his subordinates, but holding them to a strict accountability in their professional work, and always ready to appreciate and give full credit for faithful service and meritorious work.

Mr. Lovett was elected a Member of the American Society of Civil Engineers on May 3d, 1871.

MIRTILIANO SICARD, M. Am. Soc. C. E.*

Died March 17th, 1896.

Mirtiliano Sicard was born at Bogotá, Colombia, on July 2d, 1840. He was the son of Pedro Sicard Ponz, a Frenchman, and Maria Joséfa Pérez Bonalde, a Venezuelan.

He studied literature and philosophy at the Jesuits' College at Bogotá until 1854, and then attended the Military School in the same city, where he studied engineering.

In 1860, with the co-operation of his brothers, he founded a college at Lérida, which college was afterward successfully transferred by them to Ibagué.

In order to devote his time to commercial affairs, he some time later severed his connection with the college. Soon afterward he married Enriqueta Urdaneta.

His career as an engineer began at that time, since, in consequence of various contracts with the government, he brought from the United States iron bridges of various types and erected them over the rivers Combeima, China, Fotare, Coello, etc., works which honor the government which defrayed the cost. These works are of the first rank, and are a credit to the engineer who erected them. The bridge erected by Mr. Sicard over the Coello at the pass "El Chicoral" ranks with the best found in the Department. This has been acknowledged by competent persons, and especially by Joaquin Esguerra in his geographical dictionary, in which special mention is made of the bridges erected by Mr. Sicard.

* Memoir prepared by Julio Sicard, Esq.

Mr. Sicard was intimately connected with the improvements which were undertaken in the Department under various governments, and many of the public works were carried out under his management.

Returning from a recent trip to the United States, he brought back to Colombia the machinery for a mineral assaying office, which was established at Ibagué. Soon after, he had two iron boats constructed in the same country, which were intended for the rivers Saldaña and Magdalena. He also had several bridges constructed.

At all times Mr. Sicard was a useful member of society. He was constantly employed, and the offices of Secretary of the Treasury, Rector and Trustee of the Colegio de San Simon de Ibagué and Secretary of Public Instruction in the Department of the Tolima were successively filled by him. In this last employment he was overtaken by death, at Girardot, on the 17th of March, 1896, at a time when, by order of the Government, he was starting on the inspection of the work on the Railway of the Tolima.

Mr. Sicard was elected a Member of the American Society of Civil Engineers on January 2d, 1882.

HENRY CLAY YEATMAN, Jun. Am. Soc. C. E.*

DIED DECEMBER 20TH, 1896.

Henry Clay Yeatman, the eldest son of Colonel Henry Clay and Mary Brown Yeatman, was born in the City of Nashville, Tenn., March 2d, 1866. On both sides (mother and father) he came from that old original sturdy stock that made Tennessee famous in war and renowned in the councils of the Nation, and whose word was their bond. Colonel Yeatman was on the staff of General Leonidas Polk (the Fighting Bishop) during the late rebellion and is still living in Maury County, Tenn. Mrs. Yeatman (Mary Brown Polk) was a daughter of General Lucius Junius Polk, one of the first settlers of Maury County, Tenn., and she was also a great-niece of President James Knox Polk.

Henry Clay Yeatman, Jr., passed his early childhood at his home in Maury County, Tenn., and attended the country schools in the neighborhood until he was fourteen years old. He then went to school at Winchester, Tenn., for one year and for a time attended the University of Tennessee at Knoxville.

Leaving the University of Tennessee he went to Texas, and from April, 1883, until the following September was Rodman on the Galveston, Harrisburgh and San Antonio Railway on the location of several branches. He then attended Lehigh University, Pennsylvania, until January, 1886, and, returning to Texas, secured a position as Levelman on the Mexican International Railway in Mexico, and held this position until May, 1886. From May, 1886, until January, 1888, he was Assistant Engineer in charge of bridging and track-laying on the same road.

From 1888 to 1889 he was Civil Engineer in charge of the Sabinas coal mines in Mexico. From May, 1889, until August of the same year he was Engineer in charge of location and construction of irrigating works for the Eagle Pass (Texas) Irrigation and Water-Works Company. From August, 1889, to June, 1892, he was Assistant Engineer on the location of the Mexican Pacific Railway in the States of Tabasco and Chiapas, Mexico.

His health failing at this time, in the low, hot States of Tabasco and Chiapas, he returned for a time to the United States. In 1893 he went to work again on location on the Mexican International Railway, remaining until the fall of 1894, when he went to Guatemala, working for the contractors on a new railway project.

While in Guatemala he was directed to inspect certain contracts for piling, some 60 miles down the coast. He set out in an open boat with one companion, expecting to keep close to the shore, but a

* Memoir prepared by Samuel Killebrew, M. Am. Soc. C. E.

gale, striking the frail craft, carried it far from land, capsizing and sinking it. Catching a loose yawl that was on the boat they passed four days and nights in a drenching rain, all the while bailing the water out of the yawl, and were without food or drink until they were rescued by a passing Norwegian bark, which landed them at Pensacola, Fla., eight days later. This trip nearly cost Mr. Yeatman his life, but being of strong frame and robust constitution he soon rallied and returned to work on location on the Mexican International Railway, and there continued until September, 1896.

He was absolutely fearless, never hesitating in any emergency, regardless of the danger, but on the other hand, he seemed at all times to court danger. He was generous to a fault, dividing his last cent with friend or foe.

He always took a great interest in all that were under him and never grew tired in trying to advance them in every way possible. He was happiest when crowded to full capacity with work and was a most agreeable assistant. He spent the last few years of his life in Brownsville, Tenn., where he made a host of friends who mourned his sudden and unexpected death.

He died from a hemorrhage of the lungs, December 20th, 1896, at the Peabody Hotel, Memphis, Tenn., where he had gone to spend a few days.

Mr. Yeatman was elected a Junior of the American Society of Civil Engineers on May 7th, 1890.

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OF THE

American Society of Civil Engineers.

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